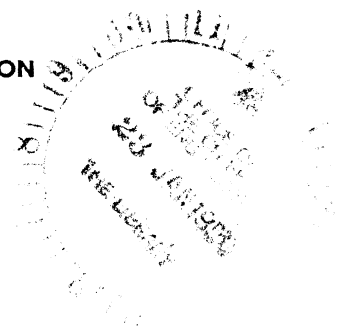


CORPS OF ENGINEERS, U. S. ARMY

**RELIEF WELL SYSTEMS FOR
DAMS AND LEVEES ON PERVIOUS FOUNDATIONS**

MODEL INVESTIGATION



TECHNICAL MEMORANDUM NO. 3-304

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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PREFACE

In 1940 the Mississippi River Commission with the approval of the Office, Chief of Engineers, initiated an investigation of the causes of and methods for controlling underseepage and sand boils along the Lower Mississippi River levees. As a part of this investigation the Waterways Experiment Station during the period 1940-1947 performed several model tests to study the phenomena of underseepage and its control by means of relief well systems.

Engineers of the Soils Division actively connected with the four model studies involved in the investigation were: model A -- Messrs. W. J. Turnbull, S. J. Johnson, C. I. Mansur, G. E. Olson, and R. R. Webb; model B -- Messrs. Turnbull and Mansur; model C -- Messrs. C. R. Horne and C. I. Mansur; model D -- Messrs. S. J. Buchanan and W. R. Perret. This memorandum was prepared by Mr. Mansur.

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SUMMARY

Where dams or levees are underlain by strata of pervious sands and gravels, excessive seepage and sand boils frequently occur during high water. Reported herein are the results of an investigation, employing sand models, of the phenomena of underseepage and the use of pressure relief wells as a means for controlling underseepage and sand boils along the Lower Mississippi River levees. The purposes of these model studies were to study the operation of relief wells and to observe well and seepage flows and landward substratum pressures with and without relief wells in operation for various foundations, seepage entrances, and top strata. The conditions studied were those considered to represent, at least qualitatively, conditions commonly encountered in the Lower Mississippi River Valley.

In the models, relief wells with proper spacing and penetration effectively reduced excess hydrostatic pressure landward of levees or dams underlain by a pervious foundation for a wide range of seepage entrances, foundation conditions, and landward top strata. With adequate well spacing and penetration, seepage normally emerging landward of a levee (without wells) may also be materially reduced, although the total underseepage flow will be increased a certain amount. The model studies indicated the importance of insuring that the wells penetrate into the principal water-carrying strata in order to obtain efficient pressure relief. Borrow pits excavated to sand riverward of a levee were found to increase underseepage and to have a pronounced effect on the design of a relief well system. Factors influencing the operation of the well

systems in the models were: well spacing, well penetration, seepage entrance conditions, stratification of the foundation, depth and permeability of the pervious stratum, and characteristics of the landside top stratum. Although the effect of well diameter was not investigated in the models, the diameter of the wells is known to be another factor which influences the performance of a well system.

In addition to the above factors which influence the design of a well system, there are other practical considerations which must be taken into account. Some of these are design of the well, maintenance of the system, corrosion, disposal of seepage, and degree of pressure relief or seepage interception desired. These factors were not a part of the model studies reported in this technical memorandum.

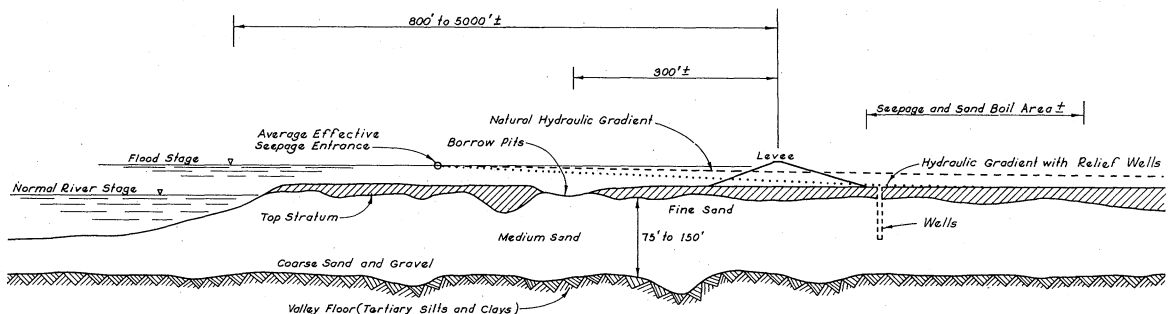
A summary of presently available formulas pertaining to the design of relief well systems, together with typical examples of their use, is given in an appendix to this report.

RELIEF WELL SYSTEMS FOR
DAMS AND LEVEES ON PERVIOUS FOUNDATIONS

Model Investigation

PART I: INTRODUCTION

1. Excessive seepage and sand boils are common occurrences during high water behind some sections of the levee system in the Lower Mississippi River Valley, especially where the levees are founded on a thin top stratum of relatively impervious soils underlain by deep strata of pervious sands and gravels. Figure 1 presents a generalization of this condition. The use of relief wells for controlling underseepage beneath dams and levees has been investigated and a number of installations have been made during the past ten years. However, most of the theoretical and model analyses made in connection with the design of pressure relief well systems have been for the case of a homogeneous pervious foundation overlain by an impervious top stratum with a vertical seepage entrance at some given distance from the line of wells. Described herein are the



GENERALIZED CROSS-SECTION OF GEOLOGIC STRATA AND LEVEES
LOWER MISSISSIPPI RIVER VALLEY

Figure 1

results of a series of model tests conducted to study the operation of relief wells for various generalized foundation conditions, seepage entrances, and landside strata, some of which are not covered by any presently available methods of analysis. Also included is a brief discussion of the conditions which cause sand boils and some of the considerations pertinent to the design of well systems for the control of underseepage. A summary of presently available formulas pertaining to the design of well systems, together with typical examples of their use, is presented as an appendix to this report.

Purpose and Scope of Investigation

2. The general purpose of these investigations was to study the operation of relief wells with various well spacings and penetrations, and to observe well and seepage flows and landward pressures with and without wells in operation for different selected conditions. Specific purposes of the investigation were:

- a. To verify available design data obtained from theoretical and electrical-model studies (see appendix), where applicable, for homogeneous foundation conditions.
- b. To obtain information on well flows and landward pressure for different well spacings and penetrations, and for various seepage entrances such as those occurring at a river a considerable distance from a levee, at the river-side toe of a levee (no riverside top stratum), and through riverside borrow pits excavated to sand.
- c. To determine the ability of a line of wells to intercept underseepage where there is little or no landside top stratum and the initial landward pressures are low without any relief wells.
- d. To determine the increase of total flow caused by wells where there is considerable natural seepage without wells (landside top stratum not impervious).

- e. To study the effect of stratification on the efficiency of a well system as regards well penetration.

The Models

3. Four sand models (designated A, B, C, and D) with different generalized foundation conditions were constructed and tested. Variables studied in these models were: well spacing and penetration; river bank, borrow pit, and open (riverside) surface seepage entrance; impervious, relatively impervious, and no landside top strata; and stratification of the foundation. The foundation and seepage conditions studied were those considered to represent, at least qualitatively, conditions commonly encountered in the Lower Mississippi River Valley. However, the results of the tests are also considered applicable, qualitatively, to other similar conditions.

4. The types of foundations studied in each model were: model A, a homogeneous sand foundation 150 ft deep; model B, a three-layer foundation consisting of a 25-ft stratum of fine sand at the top, underlain by 25 ft of medium-fine sand, which in turn was underlain by 50 ft of coarse sand, the permeability ratios being about 1:2:6.4; model C, a foundation of medium sand about 50 ft thick underlain by 10 ft of coarse sand and gravel; and model D, a two-layer foundation of fine sand 18 ft thick underlain by a stratum of very coarse sand 22 ft thick. In model A it was possible to vary the well spacing, well penetration, seepage entrance, and the landside top stratum. In model B, the well spacing and penetration were varied, open riverside borrow pits were simulated, and the distance to the source of seepage varied. In model C only the well

spacing was varied; tests were also made in this model to determine the effect on landside substratum pressures of a blanket (placed under water) over the river bank. In model D, well spacing and penetration were varied, and open landside borrow pits were simulated.

5. This report describes in detail the tests of models A and B and the results obtained. The results of the tests on models C and D, which were performed in 1941 and 1939, respectively, have previously been reported in detail in Waterways Experiment Station Technical Memorandum No. 182-1, "Seepage Model of Greenville Front Levee" and Technical Memorandum No. 151-1, "The Efficacy of Systems of Drainage Wells for the Relief of Subsurface Hydrostatic Pressures," respectively. A brief summary of the results of these latter two model studies is included in this memorandum for the purpose of comparing well performances for different foundation conditions, well screen penetration, and landside top strata.

PART II: UNDERSEEPAGE AND SAND BOILS

6. Sand boils and piping landward of levees or dams are the result of excessive hydrostatic pressures and seepage in and through pervious substrata which provide communication of pressure and seepage from the river or reservoir to landside of the structure (see fig. 1). Under certain circumstances these conditions may produce one, or a combination, of two phenomena: (1) If the subterranean pressure becomes greater than the submerged weight of the top stratum at any locality landward of a levee or dam, the excess pressure will cause heaving of the overlying soil; this may result in a concentration of seepage flow in the form of sand boils which may eventually cause failure by piping. (2) Where the foundation and top strata are heterogeneous in character, as is usually the case, seepage tends to appear at localized spots instead of causing the entire top stratum to heave or become "quick." Such seepage may start a process of subsurface erosion, or piping, that may culminate in the formation of a passage or pipe beneath the structure without any heaving action. Terzaghi has stated that the mechanics of this latter type of piping defy a theoretical approach.¹

7. The development of piping or sand boils due to heave may be explained by a consideration of the mechanics of quicksand. As water is forced upward through soil by a differential pressure it exerts a force on the soil in the direction of flow. For a loose or cohesionless soil

¹ K. Terzaghi and R. B. Peck, "Soil Mechanics in Engineering Practice" (New York: John Wiley and Sons, 1948).

the only downward force to resist this upward seepage force is the submerged weight of the soil. Thus, when the upward seepage force equals the buoyant weight of the soil no stress is transmitted between the soil grains. The mass then loses its stability and behaves like a liquid. When this occurs, sand boils or piping results. The hydraulic gradient required to cause heaving or flotation is called the "critical hydraulic gradient" and is usually about 0.7 to 1.0. (The critical hydraulic gradient is the ratio of the submerged unit weight of the soil to the unit weight of water.) Any tendency for the hydraulic gradient to increase above this only causes further expansion of the soil with increased percolation thus creating a condition favorable for subsurface erosion. If such localized erosion is allowed to continue, piping may develop beneath the dam or levee. In general, continuous "pipes" rarely form through or under a levee, but their partial formation results in progressive collapse of the soil and accelerated erosion which may ultimately cause a blow-out under the structure. Where the top stratum over a pervious foundation directly under a levee is very thin, excess pressures in the foundation may also saturate the landside toe of the levee, causing the lower portion of the embankment to slough.

8. The amount of underseepage and excess hydrostatic pressure which may develop landward of a levee depends upon the river stage, location of seepage entrance, thickness and perviousness of the landside top stratum, and carrying capacity of the pervious substratum. If there were a completely impervious landside stratum overlying the pervious foundation and there were no flow of water landward, the hydrostatic pressure head beneath the landside blanket would equal the river stage.

This condition seldom exists, as there is generally some water flowing landward through the pervious foundation and upward through the surface stratum with a consequent landward decrease of pressure head (see fig. 1). This dissipation of head was observed in model A (fig. 14) and partially explains why sand boils have not occurred in some areas where, without such head loss, subsurface pressures during high-water stages would have been sufficiently great to cause heaving or sand boils in the landside blanket.

9. The reduction of the pressure beneath the top stratum as a result of natural seepage should not be considered as a guarantee against sand boils or subsurface erosion. In nature, seepage may concentrate in cracks, root holes, or other weak spots in the top stratum to such an extent that the discharge will be sufficient to cause erosion of the underlying sandy materials. When conditions are favorable, a tunnel may gradually be created beneath the structure along a path of maximum hydraulic gradient. This is especially true if the top stratum is a cohesive material and the underlying soils are susceptible to erosion.

10. The conditions which cause sand boils to become serious are not completely understood but it is known that they depend upon several factors. Some of these factors are the increase of river stage above that required to start piping, the velocity of the seepage emerging from the boils or "pipes," and the existence of a stratum of fine, cohesionless soil readily susceptible to piping. When the pressure head which has caused a sand boil is sufficiently dissipated by the increased flow through the foundation, erosion is usually slow. However, when

there is enough pressure and the supply of water from the pervious strata is sufficient, the piping may develop through the fine sands which frequently exist immediately beneath the impervious surface stratum.

PART III: CONTROL OF UNDERSEEPAGE WITH RELIEF WELLS

11. From the preceding discussion it may be seen that the prevention of sand boils and the control of underseepage require some measure that will reduce the excess pressure beneath the landside top stratum to a value less than the submerged weight of the soil. One means of accomplishing this is to tap the underlying pervious strata with a properly designed series of wells which will provide pressure relief and controlled seepage outlets that offer less resistance to flow than any other path and at the same time prevent erosion of the foundation soils.

12. The primary requirements of a relief well system for the control of excess pressures due to underseepage are:

- a. The wells should penetrate into the principal water-carrying strata and be spaced sufficiently close together to intercept the seepage and reduce the pressure which otherwise would act beyond the wells.
- b. The wells must offer little resistance to water flowing into and out of them; they must prevent infiltration of sand into the well after initial pumping; and they must resist the deteriorative action of the water and soil.

13. Where the foundation consists of stratified deposits of pervious materials, relief wells offer several advantages as compared to gravel toes, pervious blankets, or other surface drainage measures in that they penetrate into the more pervious strata in which pressure relief is desired. For more detailed and comprehensive information on the design and installation of wells for the control of underseepage, reference is made to an article by T. A. Middlebrooks and W. H. Jarvis entitled "Relief Wells for Dams and Levees," and accompanying discussions published in Transactions (ASCE) Vol. 112, 1947, and to a report entitled

"Conference on Control of Underseepage" published by the Waterways Experiment Station in April 1945. Information on the design of well screens is contained in Waterways Experiment Station Technical Memoranda No. 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells," No. 3-250, "Investigation of Wooden Well Screens for Grenada, Enid, and Sardis Dams," and No. 3-287, "Corrosion of Drainage Wells at Sardis Dam, Mississippi."

PART IV: THEORETICAL ANALYSIS OF RELIEF WELL SYSTEMS

14. A method of analysis frequently used in the design of relief well systems is one based on a mathematical solution by M. Muskat.² Muskat assumed an infinite line of equispaced wells completely penetrating a uniform, semi-infinite, pervious stratum overlain by a uniform, absolutely impervious top stratum and underlain by a horizontal impervious stratum. The seepage entrance was assumed as a vertical plane parallel to and at a given distance from the line of wells. An analysis for the case of wells partially penetrating into the pervious stratum has been accomplished by electrical analogy models.³ The results of Muskat's analysis for fully penetrating well and Jarvis' analysis for partially penetrating wells are shown graphically in figure A4 of the appendix to this report.

15. The formulas for the generalized conditions assumed by Muskat and Jarvis are not always applicable to field conditions. For example, in nature the landside blanket is usually slightly pervious, or in other cases the seepage may enter the pervious foundation through the top stratum or open borrow pits riverward of a levee. In some cases the landward or downstream blanket may be of finite length as a result of landward borrow pits which extend through the top stratum, and in other cases there may be an impervious deposit which blocks the landward extent

² Morris Muskat, "The Flow of Homogeneous Fluids through Porous Media" (New York: McGraw-Hill Book Co., Inc., 1937)

³ W. H. Jarvis, "Mississippi River Levees, Seepage; Results of Studies at Black Bayou, Miss., Sta. 3140-3240" (Vicksburg: Vicksburg District, CE, 1939)

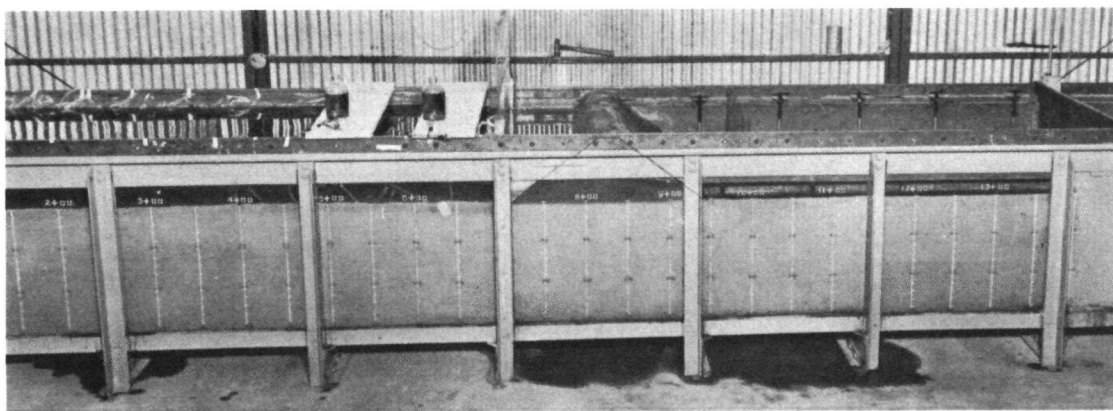
of the pervious foundation. Many combinations of conditions exist which cannot be covered specifically by theory.

16. Approximate formulas for the design of well systems have been developed by Jervis, Barron, and Bennett that permit taking into consideration some of the conditions described in the above paragraph. A summary of these formulas together with sketches of the various foundation and seepage conditions to which they apply is given in the appendix. Sample computations, which illustrate the use of the various formulas, are also included in the appendix.

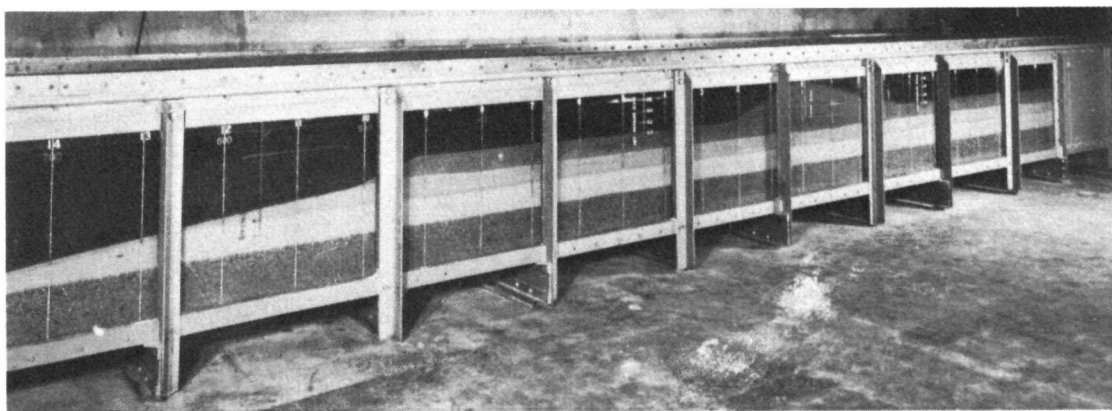
17. As previously indicated, the assumptions made in the above methods of analysis are seldom realized in the field. Therefore, the sand model tests described in subsequent paragraphs were performed not only to study the general uses of wells for controlling underseepage but to obtain information for the design of well systems for various generalized foundation conditions not subject to the above-mentioned formulas. The results of these model tests will, of course, not be directly applicable to the design of any specific well system. However, it is believed that the results obtained may be of use in a qualitative sense, and to some extent quantitatively, where the theoretical formulas are not applicable provided proper corrections are made for the conditions at hand and good engineering judgement is exercised.

PART V: DESCRIPTION OF MODELS

18. The model studies were all conducted in a steel flume approximately 28 ft long, 4 ft high, and 3.5 ft wide (see fig. 2). One side of the flume was tapped at numerous points so that piezometers could be attached for measuring pressures beneath the top stratum and within the foundation (see fig. 3). The other side of the flume was of 1/2-in.-thick plate glass, which permitted the use of dye lines for tracing flow patterns. A concentrated solution of fluorescein dye was used to trace flow lines in model B.

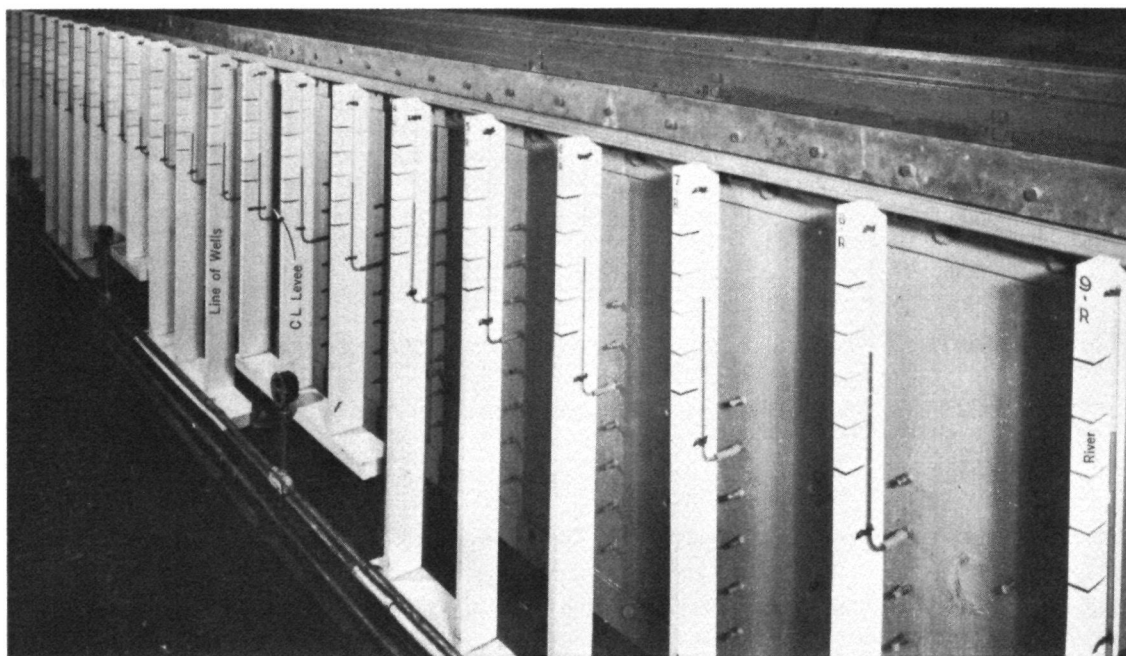


MODEL A



DEMONSTRATIONAL MODEL

Figure 2



PIEZOMETERS TO MEASURE SUBSTRATUM PRESSURES -
WELLS IN OPERATION

Figure 3

19. The primary requirements of similitude for sand models are that the flow be laminar, the model be an undistorted geometrical reproduction of the prototype, and that the permeabilities of the various strata have the same ratio as the permeabilities in nature. The absolute permeabilities of the strata only affect the rate of seepage; they have no effect on the pressure distribution within the foundation. Actually, the foundation sands used in seepage models A and B were more permeable than indicated on figures 4 and 7. The seepage flows plotted for these models were obtained by multiplying the measured model flows by the ratio of the permeabilities shown for models A, B, and C to the actual permeabilities in the respective models. As may be noted on figures 4, 7, and 9, permeabilities (k) for the sand strata in these models were chosen so that the average k_H would equal about 500×10^{-4} cm/sec.

20. Variable elevation overflow funnels were used to control the elevation of the water riverside of the levee. Similarly, shallow pans or circular overflow weirs were used to maintain a constant tailwater elevation landside of the levee. The distribution of the hydrostatic pressure within the pervious substratum for the various test conditions was obtained from the piezometers previously referred to. The discharge from the wells and seepage through the landside top stratum were measured separately in model A. All flow and head measurements in the models have been adjusted to prototype units in this report. Well and seepage flows are given in gpm per ft of net head, and hydrostatic pressures are reported as per cent of the total net head. All flow measurements have also been adjusted to a temperature of 20° C. The average temperature of the seepage water was obtained from readings taken in the forebay, in the wells, and in the tailwater.

21. The relief well systems were simulated in the models by a series of wells parallel to and approximately 25 to 50 ft landward of the levee toe, spaced so that different well spacings could be obtained by plugging various combinations of wells. The wells were so spaced that the sides of the flume always represented a plane midway between the wells, regardless of the well spacing. The wells in the model consisted of 1/2-in.-diameter copper tube riser pipe and a monel or brass screen fully penetrating the pervious foundation. Various penetrations of well screens into the pervious foundation were obtained by filling the screens to the desired elevation with sand, which subsequently could be removed with a small suction hose. The wells could be plugged with rubber stoppers fitted with small glass tubes to serve as piezometers (see fig. 10).

22. The sand strata in all the models were placed under water in order to avoid the inclusion of air with the sand. The sand was placed in thin layers through a shallow depth of water in order to minimize segregation, and was stirred and rodded to break up stratification within any given stratum of sand.

23. In analyzing the test data from the models it was necessary to make certain adjustments for the tailwater and the hydraulic head losses in the wells. The net head acting in any given model run was computed from the following equation:

$$h \text{ (net head)} = \text{elev of water RS levee} - \text{elev tailwater} - \text{friction loss in well} - \text{veloc head loss in well.}$$

The friction loss (h_f) in the wells was obtained from calibration curves of "flow vs head loss" determined experimentally in the laboratory for the riser pipe and well screen. The lengths of the riser pipes were constant in any given model; however, the "effective" length of the well screens varied depending upon the penetration into and stratification of the foundation. (For 100 per cent penetration in model A, the "effective" length of the well screen used in computing the friction loss was taken as 1/3 the total length of the screen. For penetration less than 100 per cent the "effective" length of screen was taken as 1/3 to 3/4 of the length of screen penetrating into the sand, depending upon the per cent penetration and stratification of the pervious stratum.) The velocity head loss in the riser pipes was computed from the formula, $h_v = \frac{v^2}{2g}$.

24. The hydrostatic pressure at any given point within the pervious stratum was computed from the following equation:

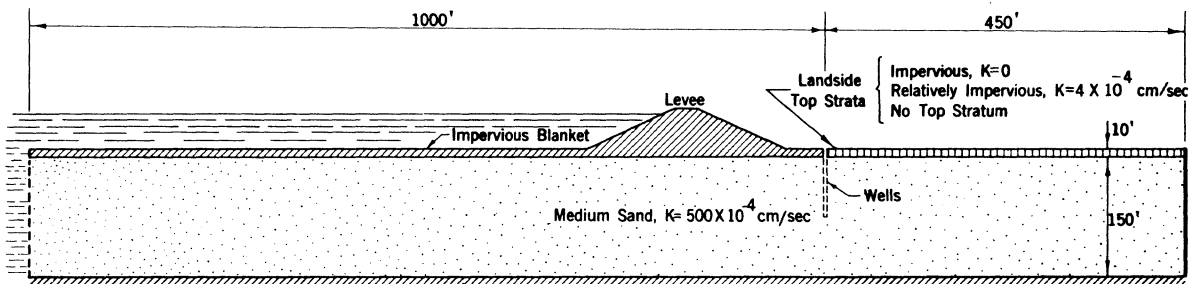
$$p = \frac{\text{Piezometer reading} - \text{tailwater} - h_f - h_v}{\text{Net head (h)}} \times 100 \text{ (in per cent)}$$

The well and seepage flows were adjusted to a net head of 1 ft (prototype) by the following equation:

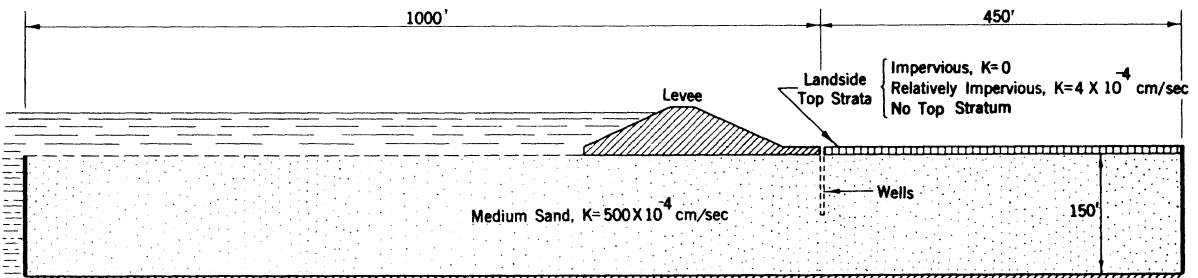
$$Q (h = 1 \text{ ft}) = \frac{Q_{\text{model}}}{h_{\text{model}}}$$

Model A -- Homogeneous Foundation

25. The first foundation condition tested for model A consisted of a homogeneous foundation of sand 150 ft deep with an impervious top stratum from the levee to the river, 1000 ft distant from the line of relief wells (model A-a, fig. 4). Three different landside top strata



MODELS A-a-1, 2, 3
Seepage Entrance at River



MODELS A-b-1, 2, 3
Seepage Entrance at Riverside Toe of Levee

MODEL A HOMOGENEOUS FOUNDATION MODEL

Figure 4

were simulated: an impervious stratum; a relatively impervious top stratum 10 ft thick with $k = \text{approximately } 4 \times 10^{-4} \text{ cm/sec}$; and no top stratum -- in other words, the foundation sand extended to the surface. In all cases the foundation extended 450 ft landward of the line of wells.

26. The dimensions of the prototype and model are given below, the scale ratio being 1:75.

<u>Dimension</u>	<u>Prototype</u>	<u>Model</u>
Depth of foundation	150 ft	2.00 ft
Distance from river to riverside toe of levee	700 ft	9.33 ft
Distance from river to wells	1000 ft	13.33 ft
Distance from wells to end of model	450 ft	6.00 ft
Total length of model	1450 ft	19.33 ft
Length of dam, or levee, or width of model	260 ft	3.50 ft
Height of dam or levee	50 ft	0.67 ft

27. The model was constructed with a forebay separated from the sand foundation by a pervious screen. This forebay represented the river in the A-a tests, in which there was an impervious top stratum from the levee to the river. The impervious riverside top stratum in these tests was obtained by placing a sheet of plastic cloth over the top of the sand to form a basin which was subsequently filled with water to an elevation higher than the water in the forebay which represented the river. The excess head of water on the plastic cloth held it firmly against the sand foundation, thereby preventing any seepage between the plastic cloth and the sand. In model A-b, the condition of no riverside top stratum was obtained by simply removing the plastic cloth.

28. The relatively impervious landside top strata of models A-a-2 and A-b-2 were simulated by means of a 1/8-in. plastic plate perforated with 0.025-in. holes spaced on 2-1/2- by 2-1/2-in. centers. The holes gave a top stratum equivalent to a 10-ft thick (prototype) stratum with

a permeability of approximately 4×10^{-4} cm/sec. The equivalent permeability of the perforated plastic plate was computed from both leakage measurements and head loss through the plate ($k_b = 3.2 \times 10^{-4}$ cm/sec), and from formula 5b of the appendix, utilizing the equivalent effective length of the landside top stratum as determined from the hydraulic grade line (without wells) in models A-a and A-b ($k_b = 4.5 \times 10^{-4}$ cm/sec).

29. This landside top stratum, with no wells in operation, resulted in hydrostatic heads at the line of wells of 36% of the net head when the seepage entrance was at the river 1000 ft from the well line (model A-a-2) and 57% of the net head when there was no riverside top stratum (model A-b-2). The arrangement of the wells, landside top stratum, and overflow weirs for model A is shown in figure 5. Well penetrations of 25, 50, and 100 per cent, and spacings of 23.6, 43.3, 86.6, 130,



MODEL A-LEVEE, WELLS, LANDSIDE TOP STRATUM, AND OVERFLOW WEIRS

Figure 5

and 260 ft were tested in this model.

30. An impervious landside top stratum was obtained by sealing all the perforations. The absence of a landside top stratum was simulated by removing the plastic plate.

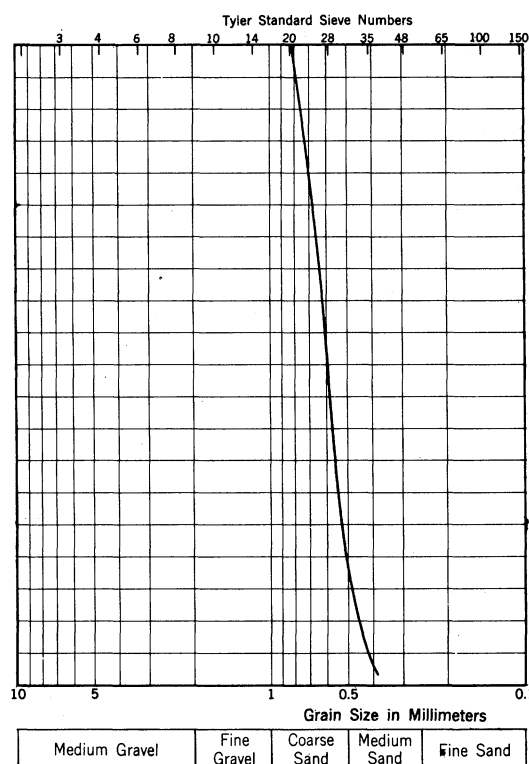
31. The second foundation condition tested (model A-b, fig. 4) was the same as described above for A-a, except there was no riverside top stratum; the pervious foundation sand was exposed at the riverside toe of the levee 300 ft from the line of wells. The same landward top strata were tested in this model as were tested in model A-a. Only a well penetration of 50 per cent was used in model A-b. The effective seepage entrance for this condition as determined from the hydraulic grade lines shown in figure 21 was about 380 ft from the line of wells.

32. The scale of model A (1:75) resulted in the diameter of the 1/2-in. well screens being equivalent to 3 ft in nature. This well diameter is somewhat larger than normally used in actual practice, which is from 3 to 24 in. The diameter of a well does not affect the well flow appreciably when the diameter is greater than 6 in.; however, it does have a somewhat greater effect on the head between wells. (For comparison of well flows and heads between wells for wells of different diameters, see table A1 of the appendix.)

33. As may be noted in figure 5, flow from the wells was kept separate from the seepage through the landside top stratum by means of a board placed immediately landside of the well line. The tailwater at the wells and over the landside top stratum was maintained at the same elevation by the constant-level weirs shown in the photograph.

34. The sand used for the pervious foundation was uniform, round

grained sand with the trade name Ottawa "Flint Shot" sand (fig. 6). The void ratio of this sand as placed in the model was 0.59. The permeability determined in the laboratory at a void ratio of 0.59 was 2300×10^{-4} cm/sec; the permeability determined in the model was 1900×10^{-4} cm/sec. A permeability of 1900×10^{-4} cm/sec was used for the sand in the model in adjusting the discharge data to a permeability of 500×10^{-4} cm/sec as shown in figure 4.



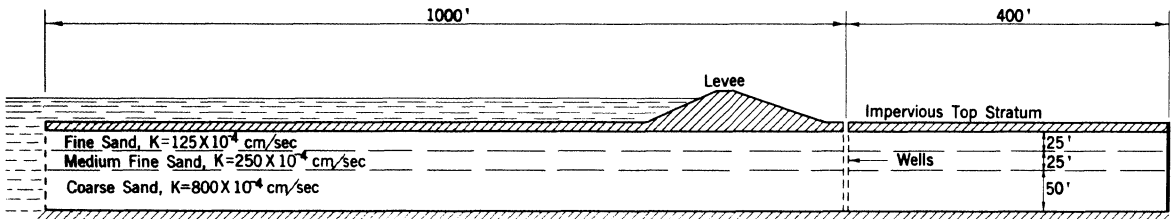
MODEL A
FOUNDATION SAND-OTTAWA "FLINT SHOT"

Figure 6

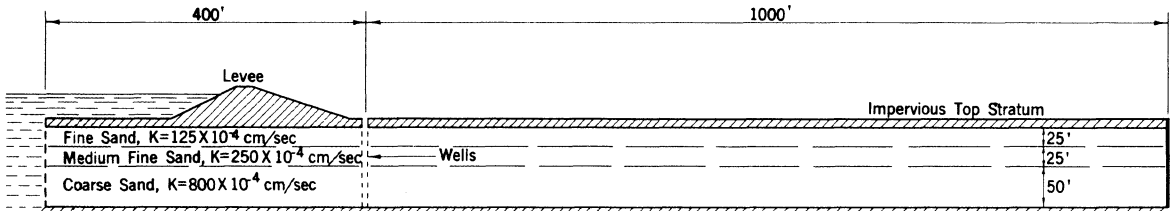
35. Since the model was to be operated over an extended period of time, precautions were taken to minimize air-locking of the sand foundation during the tests. These precautions included filtering the water through a fine sand pressure filter, heating the water to near boiling, cooling the water without exposure to air by running it through copper coils immersed in cold water, and then storing it in tanks under a 1/2-in. layer of mineral oil.

Model B -- Stratified Foundation

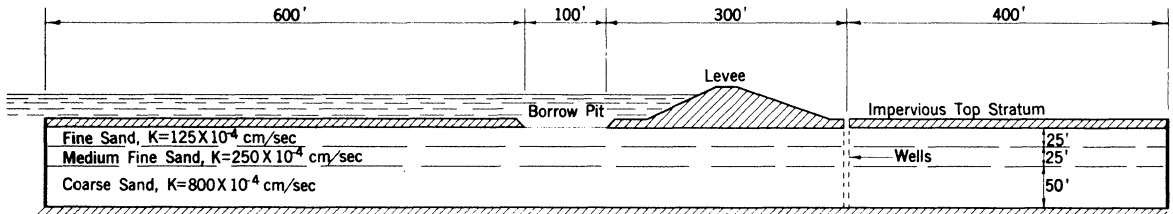
36. This model differed from model A in that the pervious foundation consisted of a 25-ft stratum of fine sand at the top, underlain by 25 ft of medium-fine sand, which in turn was underlain by 50 ft of coarse



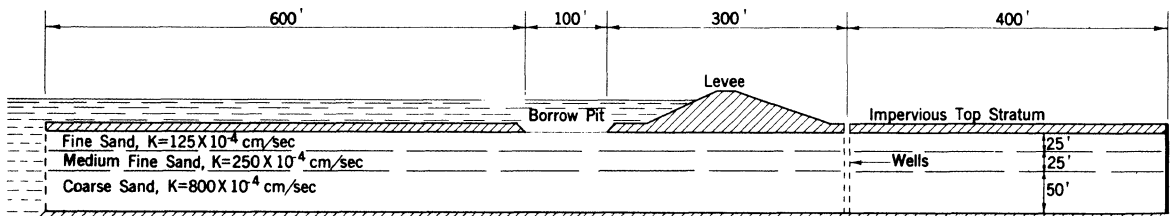
MODEL B-a
Seepage Entrance at River



MODEL B-aa
Seepage Entrance at River



MODEL B-b
Seepage Entrance in Borrow Pit Only



MODEL B-c
Seepage Entrance at River and Borrow Pit

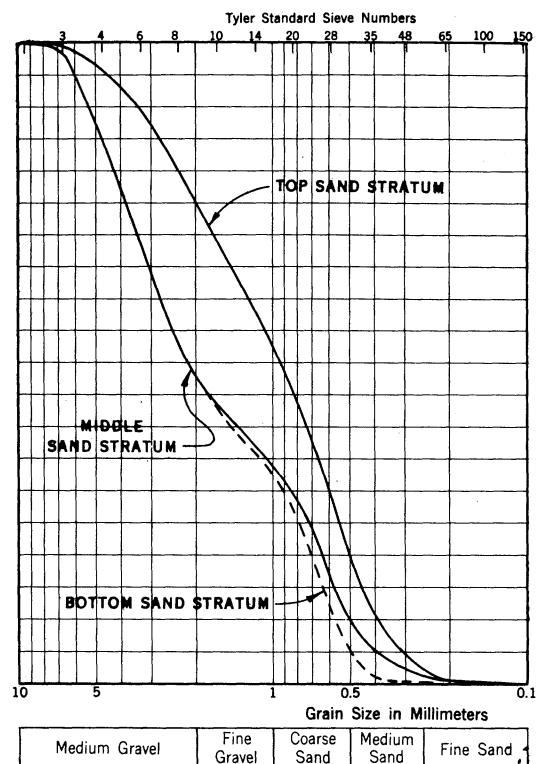
MODEL B
STRATIFIED FOUNDATION MODEL

Figure 7

sand. This particular foundation was designed to simulate foundation conditions frequently encountered beneath Mississippi River levees. The well penetrations and spacings used in this model were 10, 25, 50, and 100 per cent and 29, 58, 87, and 174 ft, respectively. Model B had a scale ratio of 1:50, which resulted in a well diameter of 2 ft. The landside and riverside top strata were impervious in all tests in model B. Figure 2 is a photograph of a demonstrational model resembling model B.

37. The primary purpose of model B was to obtain information regarding the effect of increased permeability with depth on the performance of partially penetrating wells, and the effect of riverside borrow pits excavated to sand on the design of relief well systems. The four seepage entrance conditions tested were: B-a, river 1000 ft from line of wells; B-aa, river 400 ft from line of wells; B-b, 100-ft-wide borrow pit 300 ft from wells; B-c, river and 100-ft-wide borrow pit. These models are shown in figure 7.

38. Mechanical analyses of the sands used for the various strata in the foundation are presented in figure 8. The void ratio of the upper stratum of fine sand as placed in the model was 0.39; for the medium sand, 0.42; and for the coarse sand, 0.45. The permeabilities of the



MODEL B
FOUNDATION SANDS

Figure 8

various sands as determined in the laboratory at the void ratio in the model were:

Fine sand, $k_1 = 700 \times 10^{-4}$ cm/sec; $\frac{k_1}{k_1} = 1.0$.

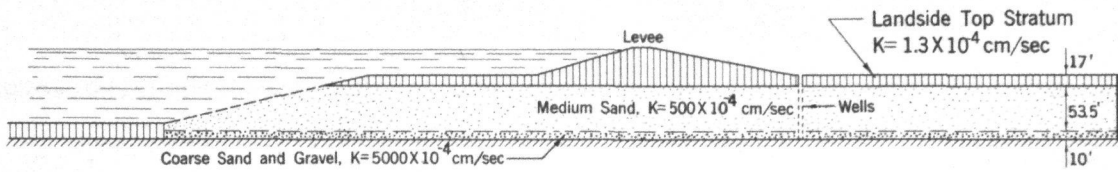
Medium sand, $k_2 = 1850 \times 10^{-4}$ cm/sec; $\frac{k_2}{k_1} = 2.6$

Coarse sand, $k_3 = 3200 \times 10^{-4}$ cm/sec; $\frac{k_3}{k_1} = 4.6$

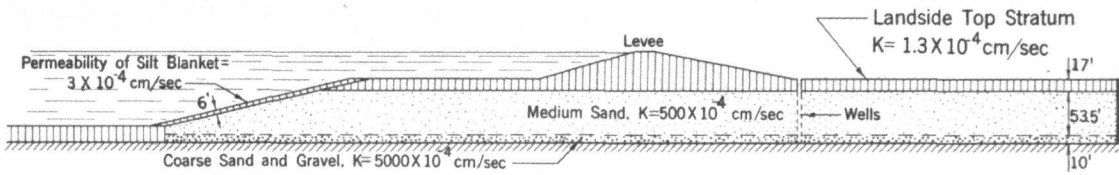
The relative permeabilities of the various strata as estimated from dye tests in the model were 1.0:1.6:7.5. The average horizontal permeability of the entire foundation as determined from tests in the model was 4000×10^{-4} cm/sec. From all of the available data permeability values of 1000×10^{-4} , 2000×10^{-4} , and 6400×10^{-4} cm/sec were assigned to the fine, medium, and coarse sand strata, respectively. In reporting the data the well flow measurements have been adjusted to an average horizontal permeability of 500×10^{-4} cm/sec for the foundation to permit comparison of data with model A. Thus, for the data reported, the fine sand stratum had a $k = 125 \times 10^{-4}$ cm/sec; the medium sand, $k = 250 \times 10^{-4}$ cm/sec; and the coarse sand, $k = 800 \times 10^{-4}$ cm/sec. The top stratum and levee in this model consisted of compacted clay silt soil which was practically impervious.

Model C -- Greenville Levee Model

39. Model C, as shown in figures 9 and 10, was constructed to simulate a foundation beneath a levee on the Mississippi River at Greenville, Miss. The foundation for this model consisted of a 17-ft top stratum of relatively impervious silt ($k = 0.8 \times 10^{-4}$ cm/sec) underlain by 53.5 ft of medium sand ($k = 300 \times 10^{-4}$ cm/sec) which, in turn, was



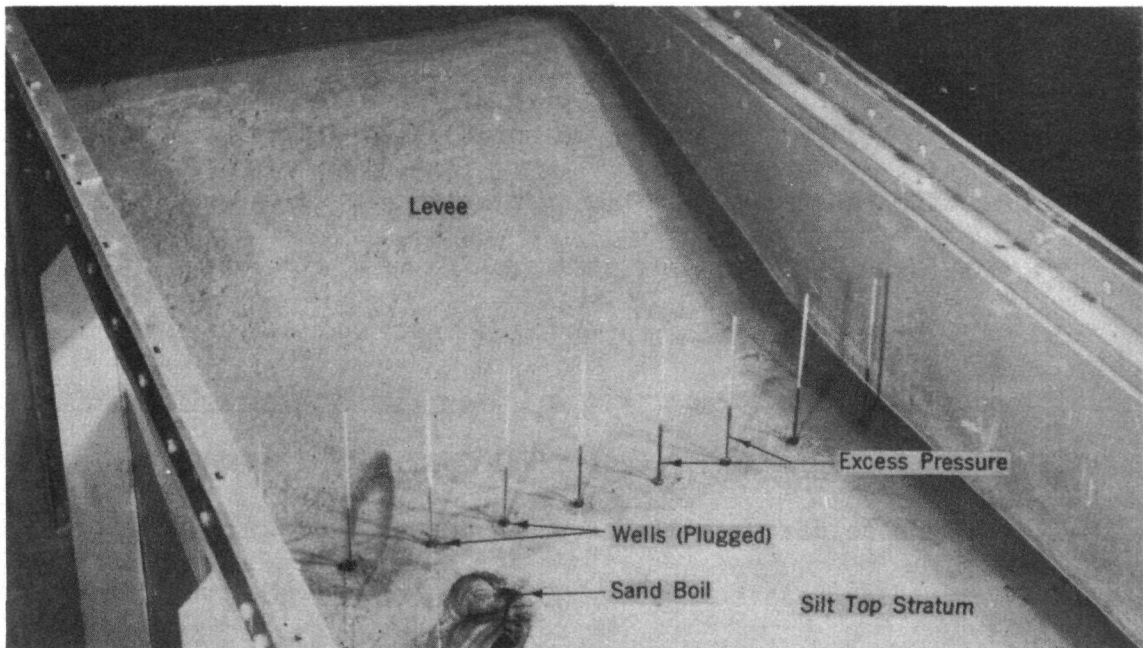
MODEL C-a
Seepage Entrance at River



MODEL C-b
Seepage Entrance at River with Silt Blanket

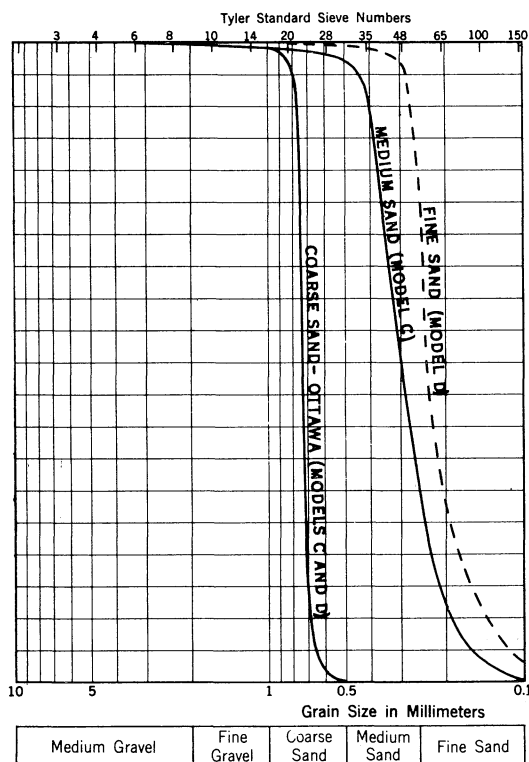
MODEL C
GREENVILLE LEVEE MODEL

Figure 9



MODEL C-EXCESS SUBSTRATUM PRESSURES

Figure 10



MODELS C AND D
FOUNDATION SANDS

Figure 11

tests with the relief wells, was also constructed of silt. Its permeability as computed from the observations of discharge and hydraulic gradients, was approximately 2×10^{-4} cm/sec. This permeability was adjusted to 3×10^{-4} cm/sec in reporting the data.

41. The tests on model C were divided into two parts, C-a and C-b. Model C-a was used to determine the effectiveness of a well system for relieving excess foundation pressures where the pervious foundation is exposed to the river approximately 600 ft from the line of wells, and the pressure which would develop beneath the natural landside top stratum without any relief wells. Model C-b was tested to determine the effectiveness of an upstream blanket (placed under water) over the exposed

underlain by 10 ft of standard Ottawa sand ($k = 3000 \times 10^{-4}$ cm/sec). Mechanical analyses of the sand used in this model are included in figure 11. In reporting the data all flow measurements have been adjusted to a permeability of $k = 1.3 \times 10^{-4}$ cm/sec for the silt blanket, $k = 500 \times 10^{-4}$ for the medium sand, and $k = 5000 \times 10^{-4}$ cm/sec for the bottom layer of coarse sand.

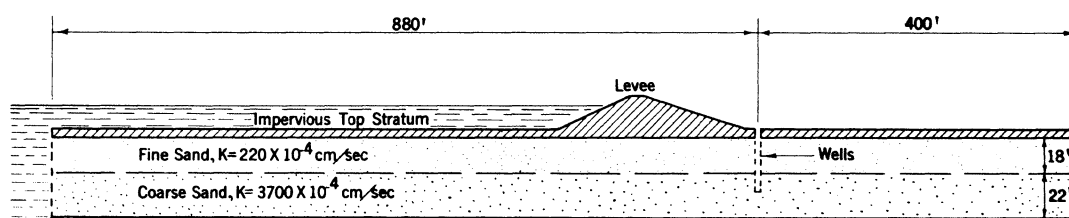
40. The impervious blanket, placed over the open face of the pervious substratum in the river channel for tests subsequent to the first

pervious foundation on landside substratum pressures.

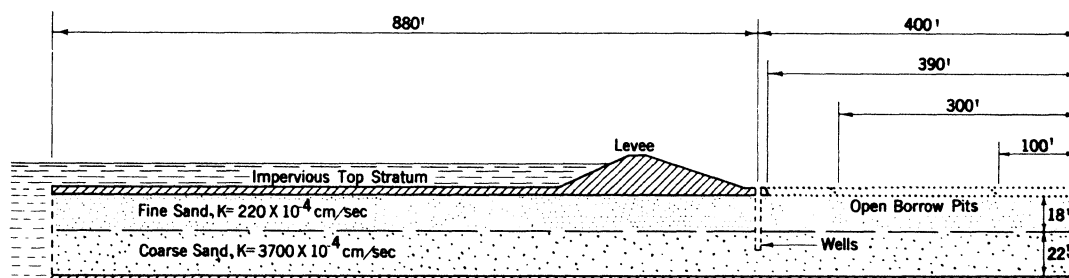
42. Well spacings of 19, 58, and 174 ft were used with a well penetration of 100 per cent. The diameter of the well screen was 2 ft; the scale of this model was 1:50.

Model D -- Memphis Levee Model

43. Model D (figs. 12 and 13) was constructed to simulate certain levees in the city of Memphis, Tenn., that are founded upon relatively thin impervious surface formations underlain by pervious strata. The top stratum in this model was 10 ft thick and had a coefficient of



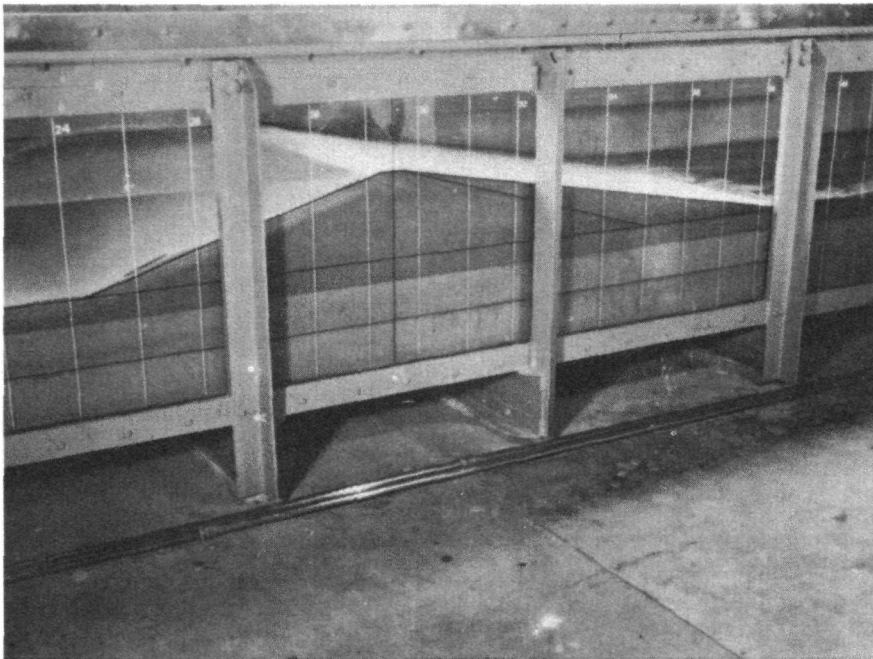
MODEL D-a
Impervious Landside Top Stratum



MODEL D-b
Open Landside Borrow Pits

MODEL D
MEMPHIS LEVEE MODEL

Figure 12



**MODEL D-CROSS-SECTION OF
LEVEE AND FOUNDATION**

Figure 13

permeability of 0.005×10^{-4} cm/sec. The underlying pervious stratum consisted of two layers -- an upper one 18 ft thick with a coefficient of permeability of 220×10^{-4} cm/sec, and a lower one with a coefficient of permeability of 3700×10^{-4} cm/sec. Mechanical analyses of the sands used in this model are shown in figure 11. In this model the source of seepage was at a vertical seepage entrance 880 ft riverward of the line of wells. The foundation extended 400 ft landward from the landside toe of the levee at which point there was an impervious barrier.

44. The experiments made in model D consisted of two principal conditions: model D-a, an impervious landside top stratum with relief wells on various spacings and with different screen openings and penetrations; and model D-b, landside borrow pits excavated to the top of

the pervious stratum with and without relief wells.

45. Well spacings of 19, 43.5, 58, 87, and 174 ft were tested in this model. The diameter of the wells was 2 ft. The scale of the model was 1:50.

PART VI: TESTS AND RESULTS

46. The tests which were performed in models A, B, C, and D, including the seepage entrances, foundation conditions, landside top strata, and well spacing and penetration are outlined in table 1. The test results are summarized in tables 2-5. The results obtained in the various models as shown in the tables and figures are in terms of prototype head (per cent) and seepage flow. All information regarding seepage entrance, foundation conditions, landside top strata, well spacing and penetration is presented on each figure showing results of the model tests. A brief discussion of the test results obtained in each model together with typical examples of performance is given in the following paragraphs. It is pointed out that all the statements made in the following sections pertain only to the particular model and wells being discussed. Application of the model results to any specific field problem would require that proper consideration be given to any change in well diameter and penetration, foundation conditions, seepage entrance, or top strata, from that tested in the model.

Model A -- Homogeneous FoundationSeepage entrance at river, model A-a

47. Model A-a-1 (fig. 14) with an impervious landside top stratum, was analyzed by the Muskat-Jervis curves shown in figure A4 of the appendix. Model A-a-2, with a relatively impervious landside top stratum, was analyzed for 100 per cent well penetration using formulas 6, 10, and 11 of the appendix. The natural seepage flow for the model with no wells

and no landside top stratum (A-a-3) was computed from the formulas given below.

$$Q = \oint kH$$

where Q = seepage flow per unit length of levee,

\oint = shape factor,

k = coefficient of permeability, and

H = head (net) on levee or dam.

$$\oint = H' \frac{K(90^\circ - \theta)}{K(\theta)}$$

where K = complete elliptical integral of first kind of modulus $(90^\circ - \theta)$ and θ , respectively,

$$H' = \frac{1}{1 + \frac{K(90^\circ - \theta)}{K(\theta)} \left[\frac{L_1}{d} \right]},$$

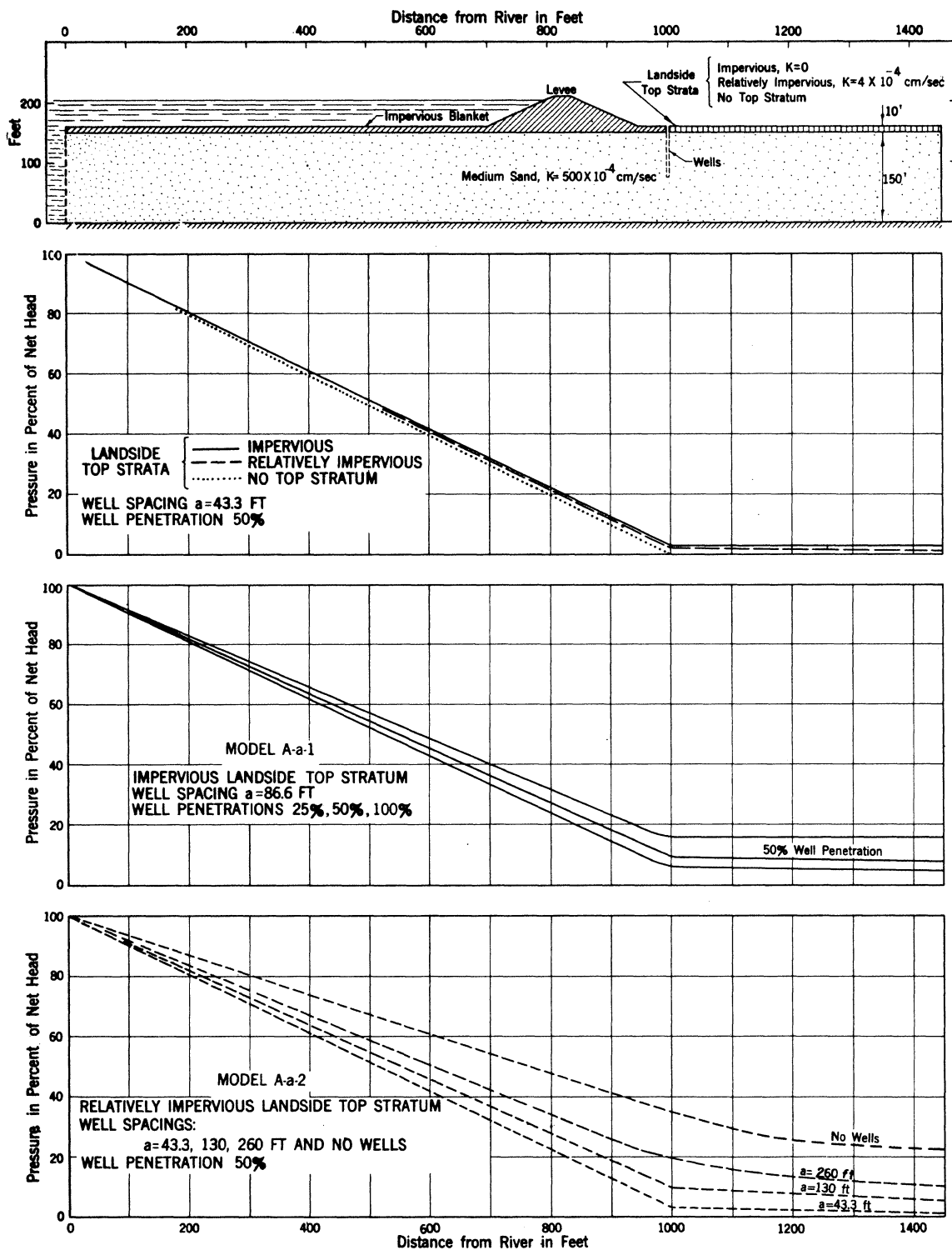
$$\sin \theta = \tanh \frac{\pi L_2}{2d},$$

L_1 = distance from RS toe of levee to river,

d = depth of pervious stratum,

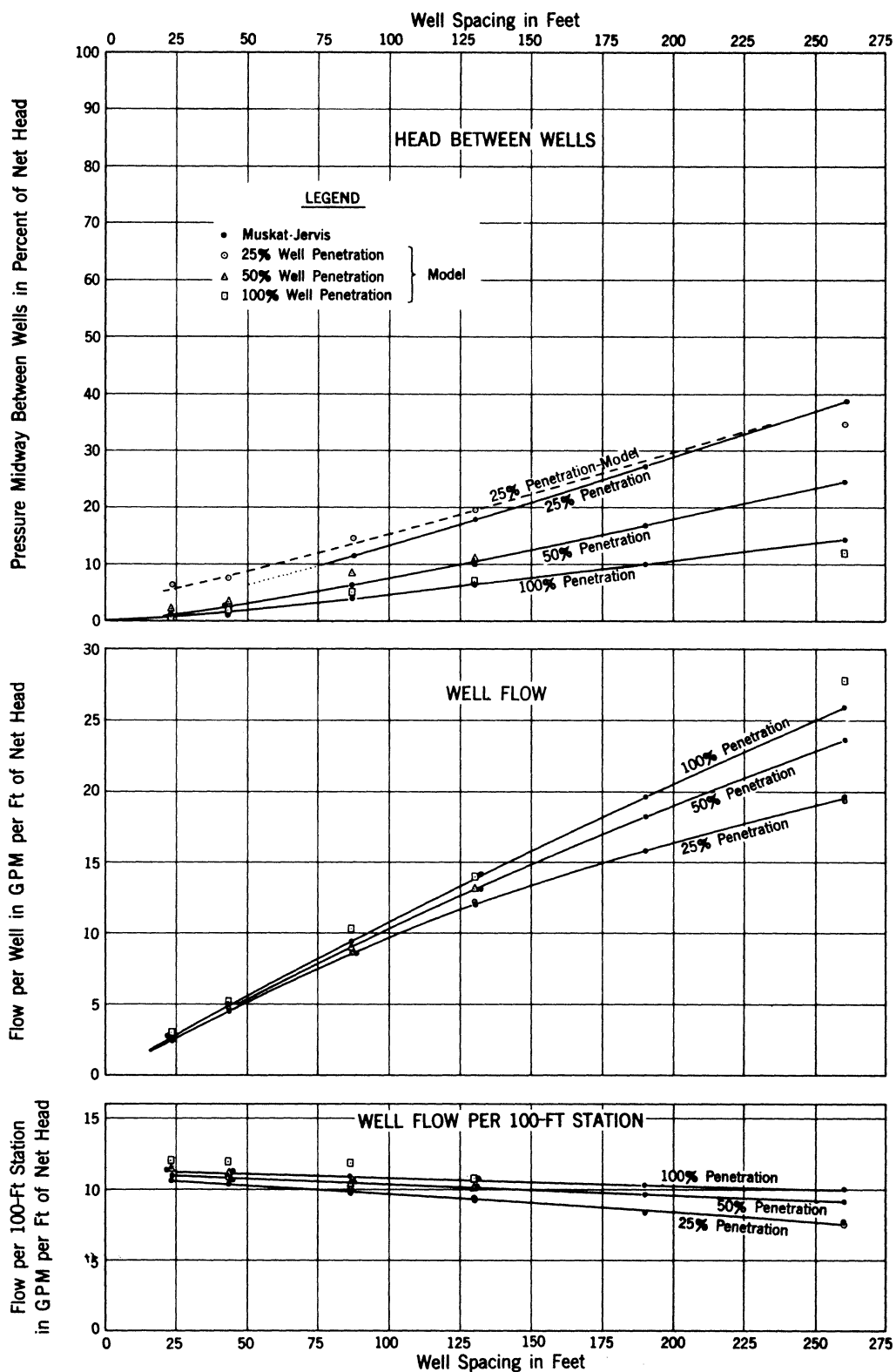
L_2 = base width of levee or dam.

48. The hydrostatic head immediately beneath the top stratum is shown for various well spacings and penetrations, and landside top strata on figure 14. Well flows and the head midway between wells at the landside toe of the levee have been plotted for model A-a-1 for various well penetrations and spacings on figure 15. The well flows and head midway between wells as computed from the Muskat-Jervis curves are also shown on this figure. Well flows and the residual head between wells as obtained in models A-a-2 (relatively impervious landside top stratum) and



MODELS A-a-1,2,3
HYDROSTATIC PRESSURES BENEATH TOP STRATUM
SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS

Figure 14



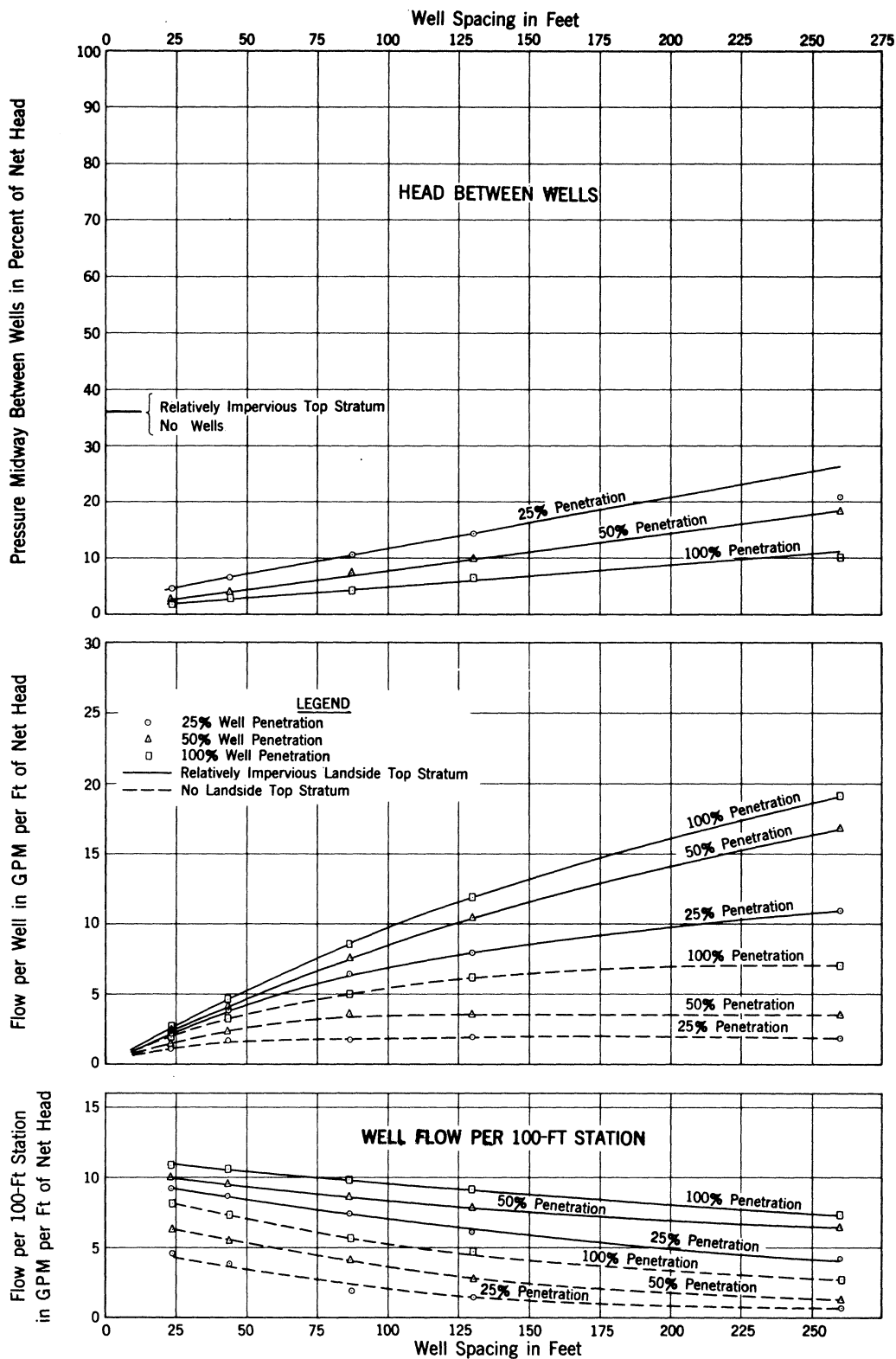
MODEL A-a-1
WELL FLOWS AND LANDSIDE
SUBSTRATUM PRESSURES
SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS
IMPERVIOUS LANDSIDE TOP STRATUM

Figure 15

A-a-3 (no landside top stratum) are shown for various well spacings and penetrations on figure 16. The effect of the landside top stratum on well flows and on the head between wells is shown on figure 17 for the three landside strata tested.

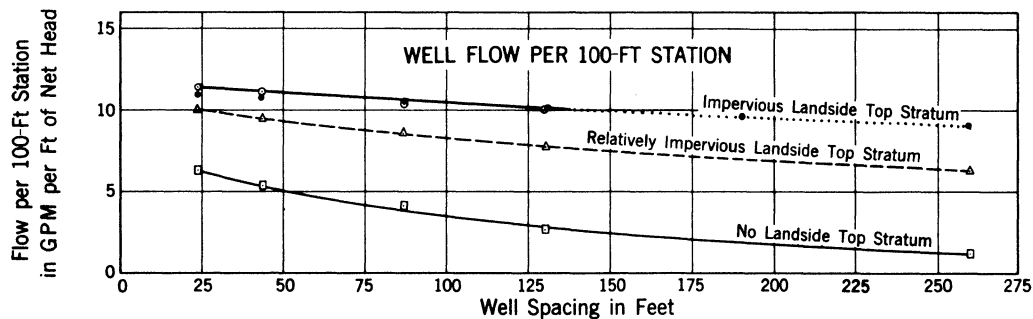
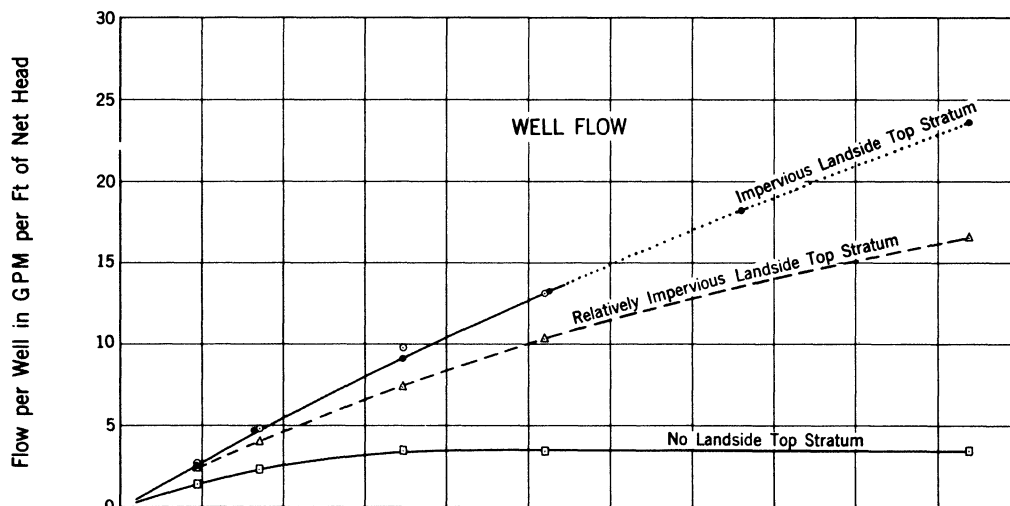
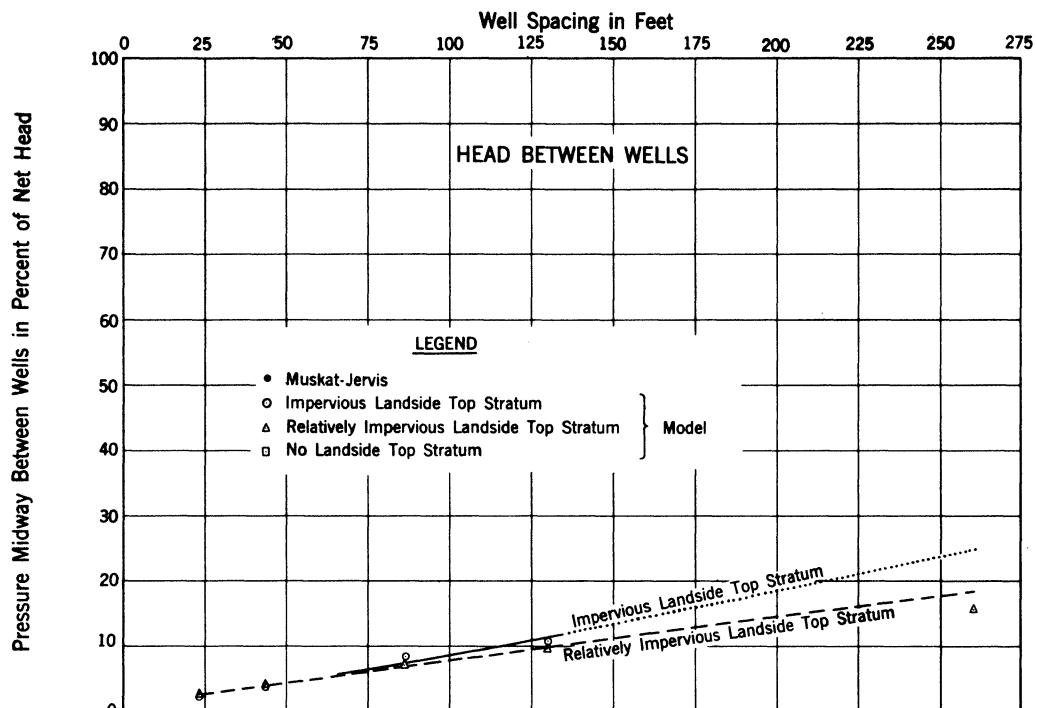
49. As previously pointed out, top strata landward of levees in nature are seldom completely impervious and usually permit a certain amount of natural seepage during periods of high water. This seepage may or may not result in a considerable reduction of pressure landward of the levee, depending upon the characteristics of the foundation and top stratum. An example of pressure reduction as a result of landside seepage⁴ is shown in the lower graph of figure 14. (The Muskat-Jervis well formulas do not take into consideration the effect of natural seepage on well flow and reduction of landside pressures.) The relationship between well flow and natural seepage is shown for models A-a-2 and -3 on figures 18 and 19. Interception of underseepage by a system of wells with an attendant reduction of natural seepage landward of the wells is also shown on these figures. The increase of flow due to wells in excess of that naturally occurring without wells is also shown for models A-a-2 and -3 on figures 18 and 19. Equipotential lines for the model with the relatively impervious landside top stratum (model A-a-2) are shown for the cases with no wells and with relief wells on approximately 25-ft centers on figure 20. A summary of all test data and analyses for model A is given in table 2. -

⁴ The term "seepage" as used in this report and on the figures for model A is limited to the seepage or water rising to the surface through the top stratum landward of the line of relief wells.



MODELS A-a-2,3
**WELL FLOWS AND LANDSIDE
 SUBSTRATUM PRESSURES**
 SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS
 RELATIVELY IMPERVIOUS AND NO LANDSIDE TOP STRATUM

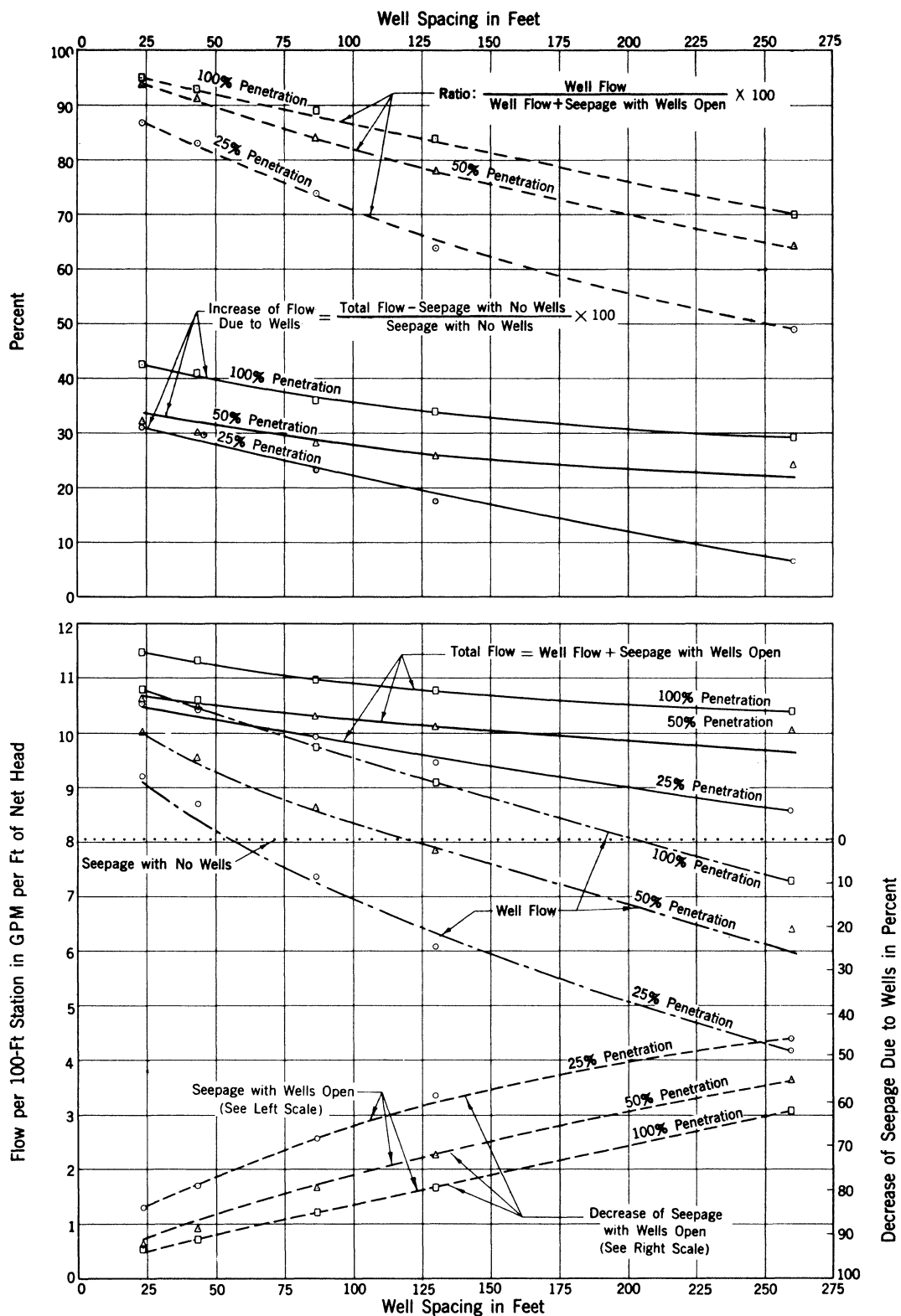
Figure 16



MODEL A-a
WELL FLOWS AND LANDSIDE
SUBSTRATUM PRESSURES

SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS— WELL PENETRATION 50%
IMPERVIOUS, RELATIVELY IMPERVIOUS, AND NO LANDSIDE TOP STRATUM

Figure 17



MODEL A-a-2

WELL FLOW AND SEEPAGE **SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS** **RELATIVELY IMPERVIOUS LANDSIDE TOP STRATUM**

Figure 18

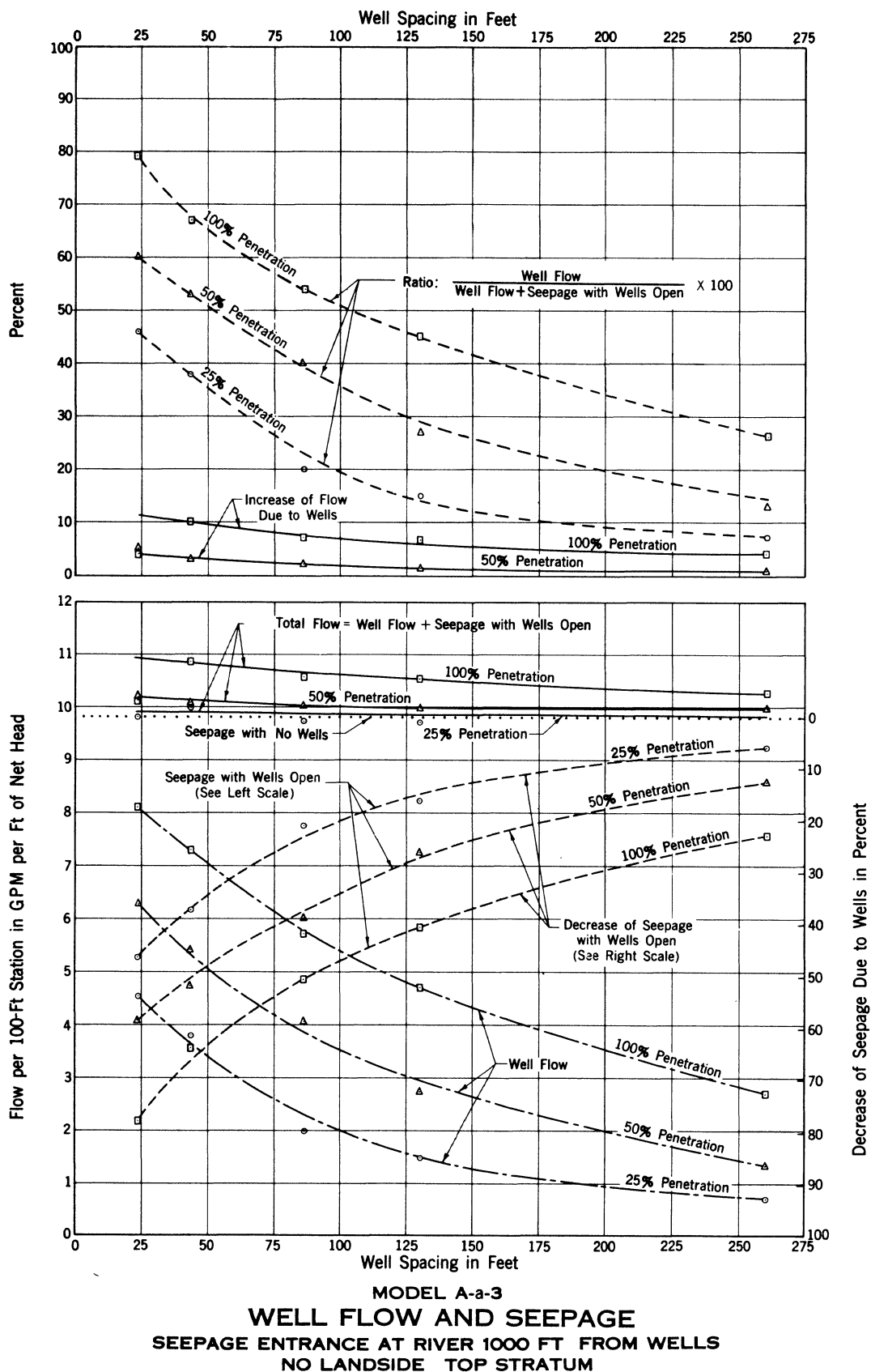


Figure 19

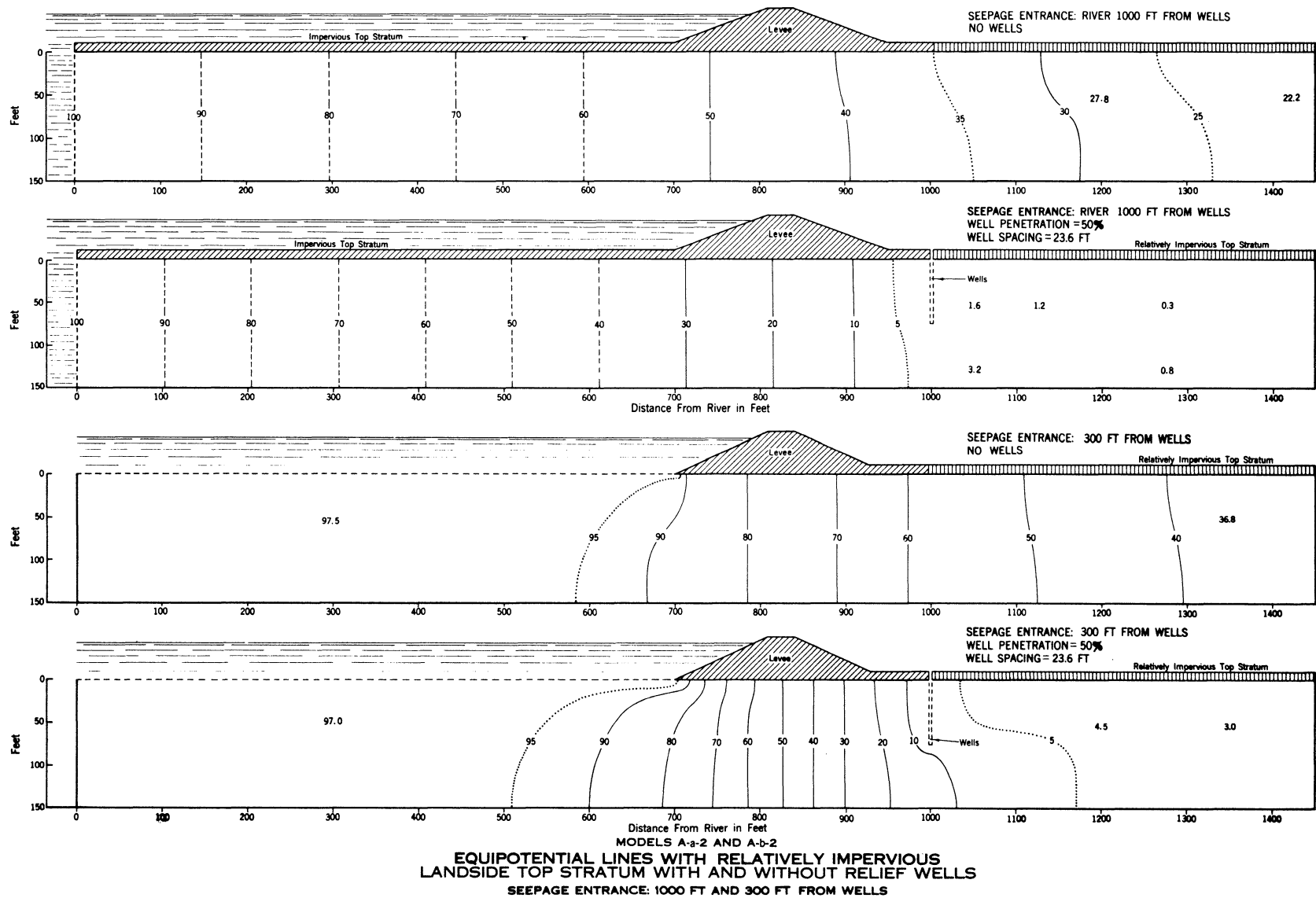


Figure 20

50. Model A-a-1. The test data presented on figures 14 and 15 show that the relief wells significantly reduced the landward pressures which otherwise would have been present for this foundation and top stratum conditions. For this model the landward pressure was reduced 90% by either of the following combinations of well spacing, a , and penetration, W : $W = 25\%$, $a = 60$ ft; $W = 50\%$, $a = 125$ ft; or $W = 100\%$, $a = 190$ ft (see fig. 15). Well spacings less than 200 ft and penetrations greater than 25% had relatively little effect on well flow per 100-ft station but did have an effect on residual pressure between the wells. As may be noted on figure 15, close agreement was obtained between the model results and the values computed from the Muskat-Jervis design curves shown in the appendix.

51. Model A-a-2. For this particular model the hydrostatic head at the landside toe of the levee was 36% of the net head on the levee with no wells in operation (see figs. 14 and 16). As may be noted from figure 16 this excess pressure of 36% H at the landside levee toe was reduced to 5% H with a well spacing of 50 ft and penetration of 50%. The natural seepage through the landside blanket with no wells was reduced approximately 80% by wells on 85-ft centers with a penetration of 50%. For this particular case the wells increased the total underseepage flow (well flow + seepage with wells open) 28% above the natural seepage which occurred with no wells (see fig. 18). The head midway between the wells and the well flow were computed for the case of fully penetrating wells using formulas 6, 10, and 11 of the appendix. (The value of L in these formulas was obtained by projecting landward the straight-line portion of the hydraulic grade line obtained in the model with no wells in

operation, from the river until it intersected the tailwater, or ground surface, landside of the levee, L being equal to the distance from the river to this point.) The results of these computations are shown in table 2. As may be noted, the theoretical results compare fairly closely with those of the model tests.

52. Model A-a-3. A condition sometimes encountered in nature is that of practically no landside blanket. As a result, little excess pressure can develop landward of the levee regardless of the river stage; however, excessive seepage rising to the surface may cause numerous small sand boils and possibly detrimental piping beneath the structure. Although wells operating under gravity flow cannot reduce the pressure at the surface any further for this condition, the test data presented on figure 19 show that such wells, if sufficiently deep and close together, will intercept a considerable proportion of the underseepage which otherwise would rise to the surface as erosional seepage. For example, in this model wells with a penetration of 50% spaced on 50-ft centers reduced the natural seepage by approximately 50% but at the same time increased the total underseepage only 3% more than the natural seepage which occurred with no wells (see fig. 19).

53. Models A-a-1, -2, -3. A comparison of well flows and corresponding head between wells for "impervious," "relatively impervious," and "no" landside top strata is shown for a well penetration of 50% on figure 17. This figure shows that for any given well system the maximum pressure between wells is that obtained for the case of an impervious landside top stratum. The head between wells will always be less for relatively impervious top strata than for impervious top strata because of the head

reduction occasioned by the natural seepage through the top strata in the former case. The well flow is a maximum for any given spacing for the case of an impervious landside blanket and a minimum for the case of no landside blanket (see fig. 17). Although the well flow is somewhat less for relatively impervious landside top strata, the total underseepage flow (well flow + seepage with wells open) is some greater than the well flow for an impervious landside blanket because of the natural seepage through the landside blanket.

54. The well and seepage flows for distances from the line of wells to the seepage entrance (river) other than 1000 ft may be computed for various well penetrations and spacings for this particular foundation and landside top stratum from the following formula. This formula is based on a straight-line variation of head beyond 300 ft riverward from the line of wells to the river.

$$Q = \frac{112,500 H k q}{q (s - 1000) + 11,250}$$

where: q = well or seepage flow in gpm per 100 ft of levee as given in table 2,

H = net head in ft,

k = coefficient of permeability in ft per min,

s = distance from line of wells to river in ft (s equal to or greater than 300 ft), and

Q = well or seepage flow per 100 ft of levee in gpm.

The hydrostatic head midway between the wells may also be computed for distances other than 1000 ft from the following formula,

$$P = \frac{150 H (\%) }{0.0133 q (s - 1000) + 150}$$

where: (%) = per cent of head between wells as obtained for $s = 1000$
(see table 2), and

P = head in ft (s equal to or greater than 300 ft).

Seepage entrance at levee toe, model A-b

55. Model A-b (figs. 4 and 21) had the same pervious foundation as model A-a but had no riverside top stratum over the pervious sand. This model simulated a condition similar to one sometimes created in nature where the riverside top stratum is removed during construction of a levee. As in model A-a three different landside top strata were simulated. The same relatively impervious landside top stratum was used in this model as was used in model A-a, but because of the closer seepage entrance the hydrostatic head at the line of wells was 57% H rather than 36% H as in model A-a. Model A-b-1, with an impervious landside top stratum, was analyzed by the Muskat-Jervis design curves shown in figure A4 of the appendix, utilizing an effective distance of 380 ft to the seepage entrance. This distance was obtained by projecting riverward the straight-line portion of the hydraulic grade lines beneath the levee until it intersected the unit head on the levee. The natural seepage flow for the model with no wells and no landside top stratum (A-b-3) was computed from the formulas⁵ given below.

$$Q = \oint k H$$

$$\text{where: } \oint = \frac{K (90^\circ - \theta)}{2 K (\theta)} \quad \text{and} \quad \sin \theta = \tanh \left(\frac{\pi L_2}{4d} \right)$$

(See paragraph 47 for definition of terms.)

⁵ Op. cit., footnote 2, p 11.

or where

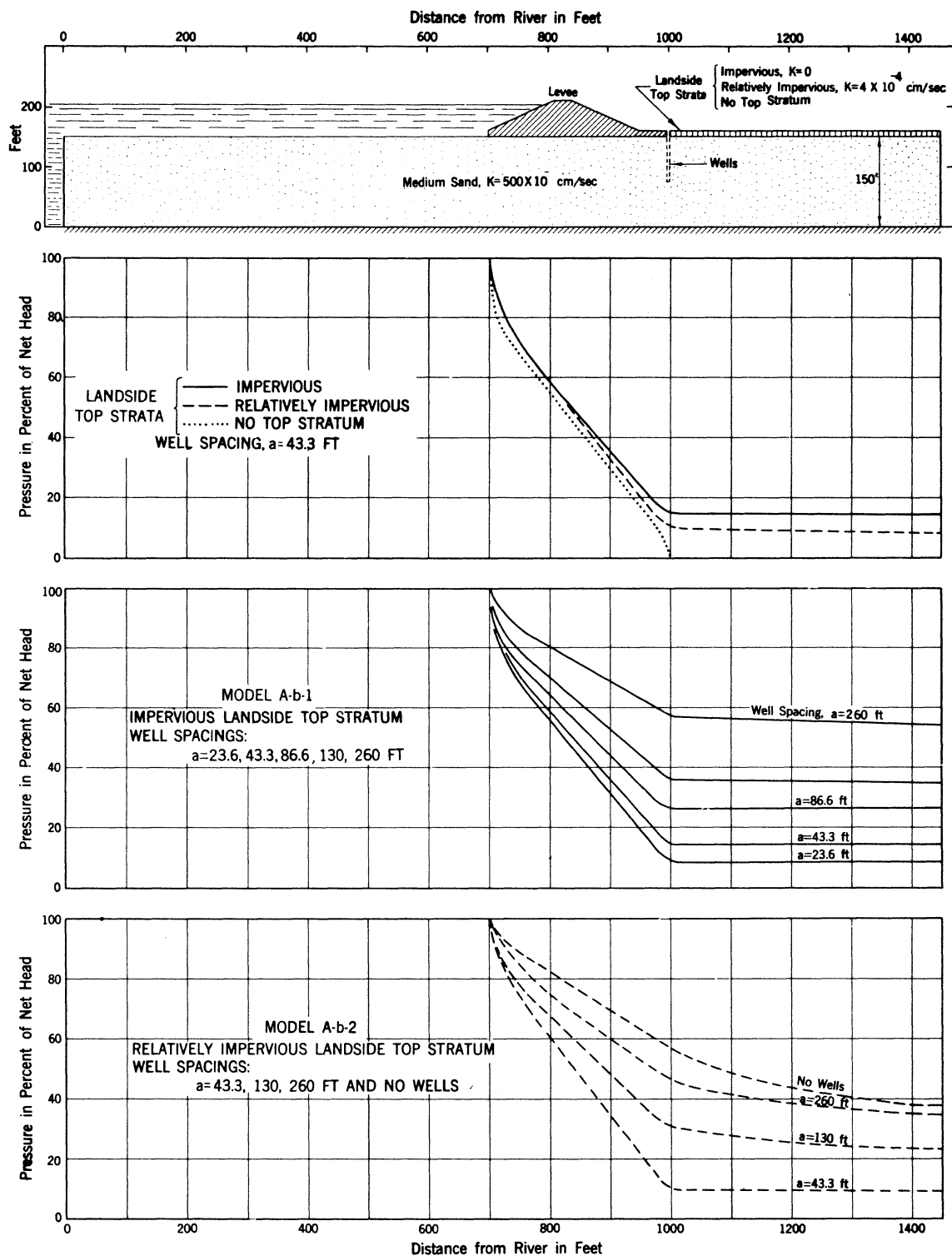
$$\frac{L_2}{d} > 1 \quad f = \frac{1}{0.86 + \frac{L_2}{d}}$$

$$\frac{L_2}{d} < 1 \quad f = 0.73 \log_{10} \frac{13 + \left(\frac{L_2}{d}\right)^2}{2.54 \left(\frac{L_2}{d}\right)}$$

56. The hydrostatic head immediately beneath the top stratum is shown on figure 21 for the various well spacings and penetrations and landside top strata tested. Well flows and the head midway between wells at the landside toe of the levee have been plotted for model A-b for various landside top strata and well spacings and a well penetration of 50% on figure 22. The results of the theoretical analyses for model A-b are given in table 2.

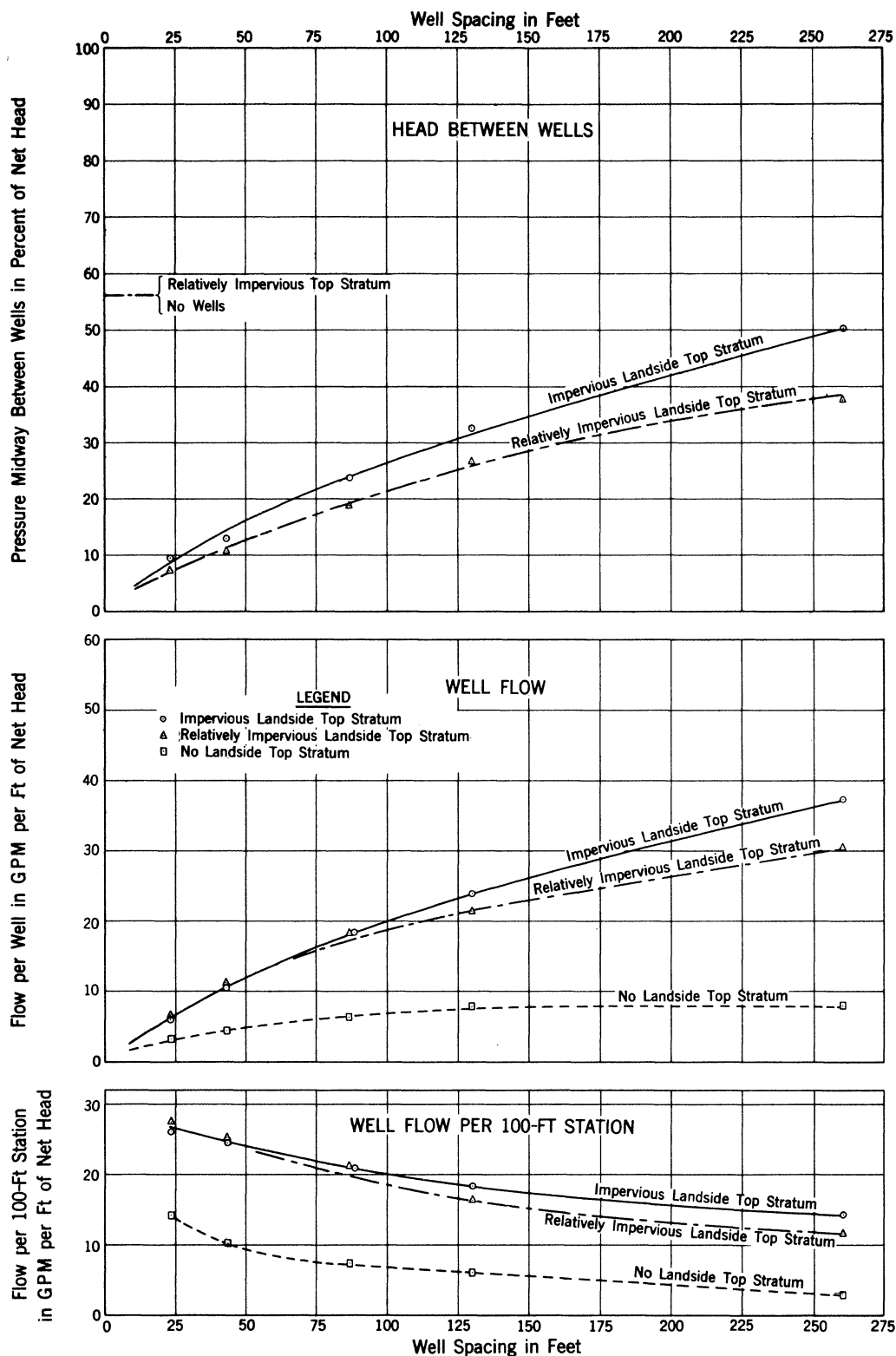
57. As observed in model A-a-2 for a seeping landside blanket, the wells in this model with a penetration of 50% did not intercept all of the seepage beneath the levee. The relationship between well flow and natural seepage for model A-b-2 and a well penetration of 50% is shown on figure 23. Interception of underseepage by a system of wells with an attendant reduction of natural seepage landward of the wells, the increase of total underseepage flow caused by the wells in excess of that naturally occurring without wells, and the proportion of the total underseepage flow intercepted by the wells are also shown on figure 23.

58. Model A-b-1. Although the source of seepage was extremely close to the line of wells, the test data presented on figures 21 and 22 show that the wells significantly reduced the landward pressure which otherwise would have been present for this foundation and top stratum. Because of the closeness of the seepage entrance the well flows in this



MODELS A-b-1-2-3
 HYDROSTATIC PRESSURES BENEATH TOP STRATUM
 SEEPAGE ENTRANCE 300 FT FROM WELLS
 WELL PENETRATION 50%

Figure 21



MODEL A-b WELL FLOWS AND LANDSIDE SUBSTRATUM PRESSURES

SEEPAGE ENTRANCE 300 FT FROM WELLS—WELL PENETRATION 50%
IMPERVIOUS, RELATIVELY IMPERVIOUS, AND NO LANDSIDE TOP STRATUM

Figure 22

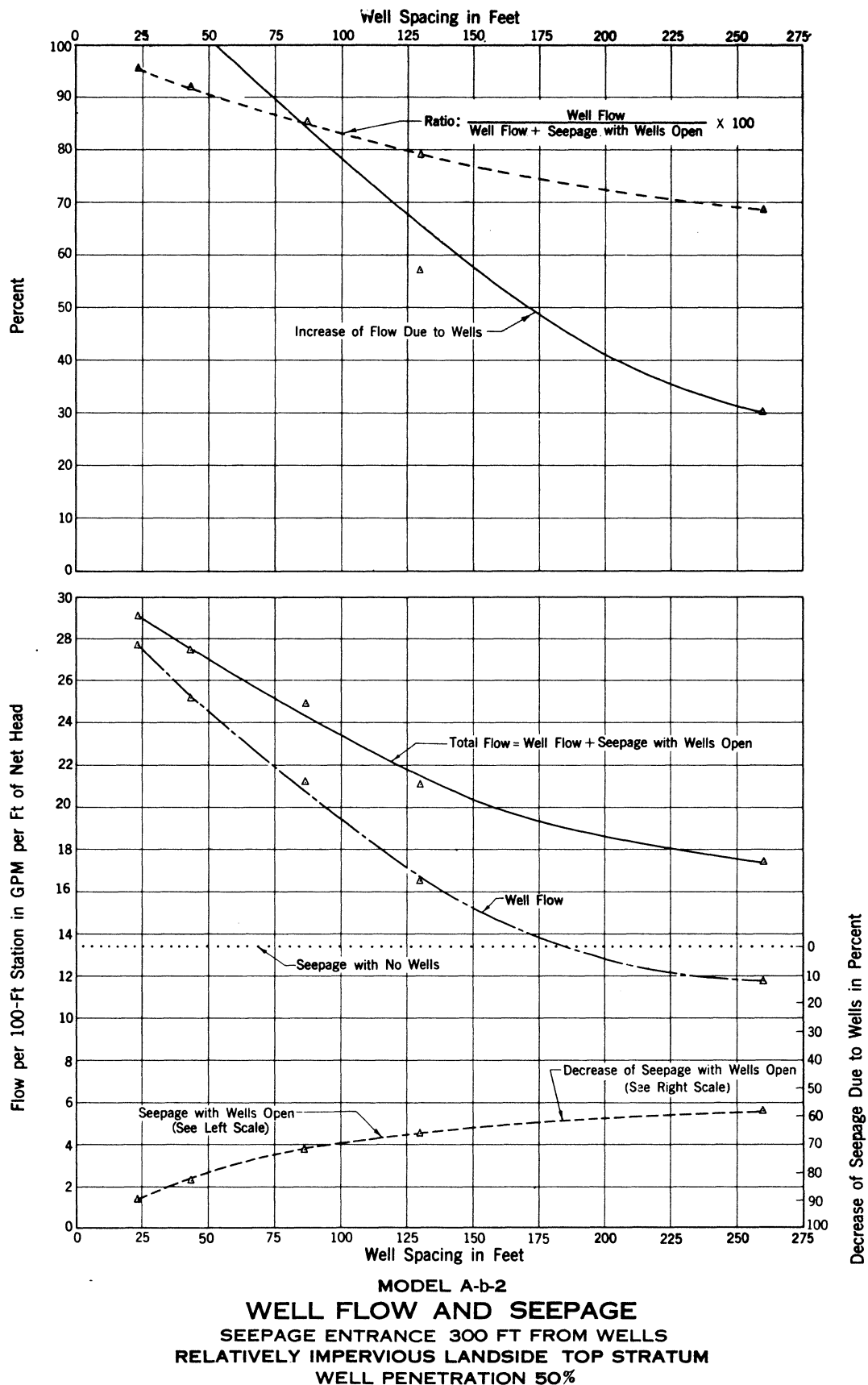
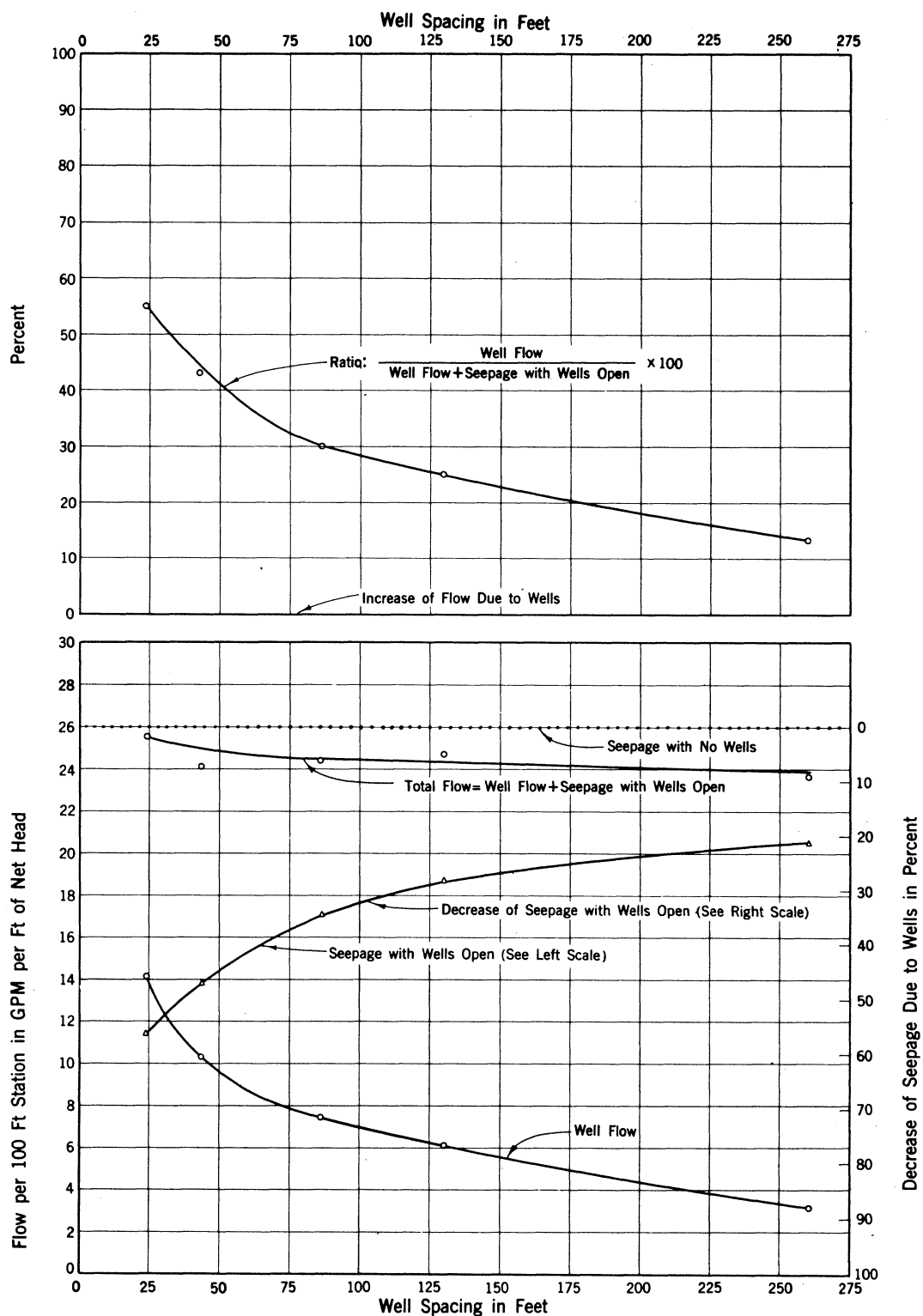


Figure 23

model (A-b-1) were considerably greater than the flows obtained in model A-a-1; for the same reason the wells in model A-b-1 had to be considerably closer together to achieve the same pressure reduction than was necessary in model A-a-1 (compare figs. 17 and 22). In model A-b-1 the head between wells was reduced approximately 85% with well spacings of 50 ft and a penetration of 50% (see fig. 22). As may be noted in table 2 reasonably close agreement was obtained between model results and the values computed from the Muskat-Jervis design curves shown in the appendix.

59. Model A-b-2. For this particular model the hydrostatic head at the landside toe of the levee was 57% of the net head on the levee with no wells in operation (see figs. 21 and 22). As may be noted from figure 22 this pressure of 57% H at the landside levee toe was reduced to 12% H with a well spacing of 50 ft and a penetration of 50%. The natural seepage through the landside blanket with no wells was reduced approximately 80% by this particular well system. For this case the wells increased the total underseepage flow (well flow + seepage with wells) 100% above the natural seepage which occurred with no wells (see fig. 23). Equipotential lines for this model are shown for the cases with no wells and with wells on figure 20.

60. Model A-b-3. With the wells in model A-b-3 sufficiently close together a considerable proportion of the underseepage was intercepted which otherwise would have risen to the surface as erosional seepage. For example, in this model wells with a penetration of 50% spaced on 50-ft centers reduced the natural seepage by approximately 45%, while at the same time there was no increase in the total underseepage which occurred with no wells (see fig. 24).



MODEL A-b-3
WELL FLOW AND SEEPAGE
 SEEPAGE ENTRANCE 300 FT FROM WELLS
 NO LANDSIDE TOP STRATUM
 WELL PENETRATION 50%

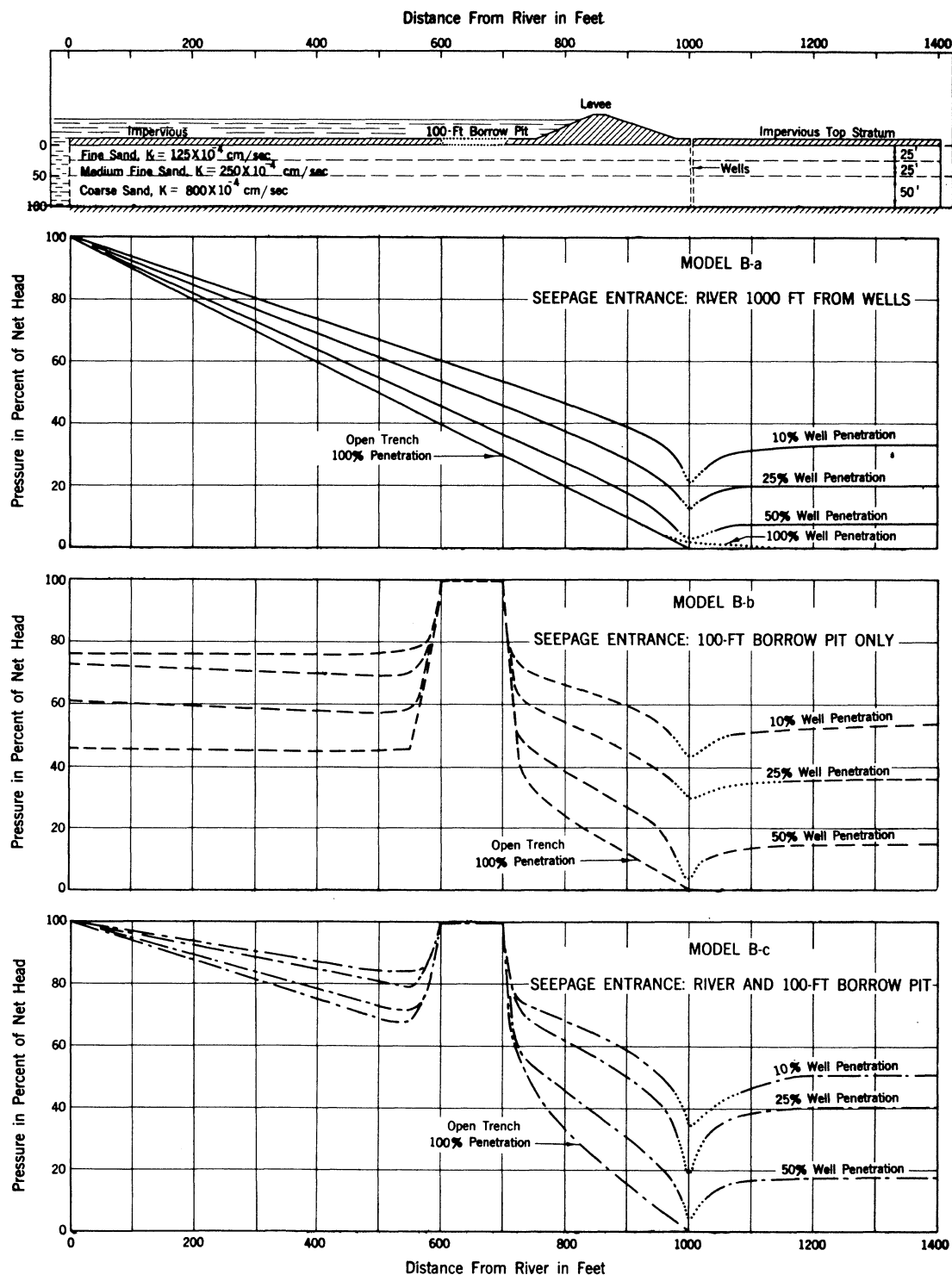
Figure 24

61. Models A-a and A-b. To obtain the same landward pressure relief for model A-b as for model A-a (assuming the same landside top stratum and well penetration) required that the wells be spaced 3 to 4 times closer than in model A-a. The effective distance from the wells to the seepage entrance in model A-a was 1000 ft while in model A-b it was approximately 380 ft. The well flows in model A-b were approximately $2\frac{1}{2}$ times those in model A-a for the same landside top stratum, well spacing, and penetration.

Model B -- Stratified Foundation

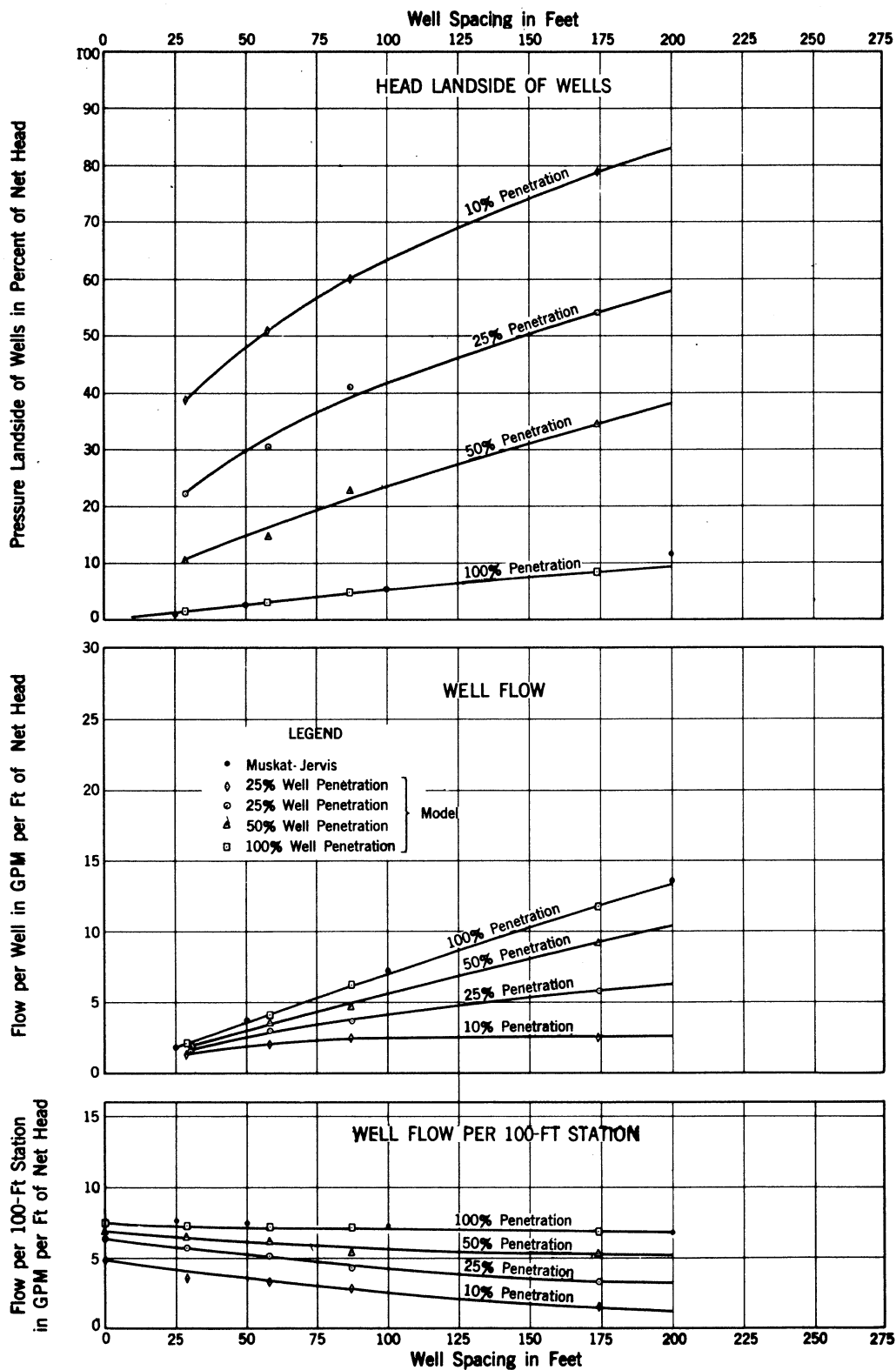
Seepage entrance at river, model B-a

62. The hydrostatic head immediately beneath the top stratum for a well of spacing of 29 ft and for the various penetrations and seepage entrance conditions tested is shown on figure 25. Hydraulic grade lines from the seepage entrance for model B-a are shown in the upper portion of this figure. Well flows and the head midway between wells at the landside toe of the levee have been plotted for this model for various well penetrations and spacings on figure 26. Well flows and heads midway between wells as computed from the Muskat-Jervis curves are also shown on this figure for the 100% penetration case. The Muskat-Jervis curves are not applicable to partially penetrating wells in stratified foundations. A similar plot of well flows and pressures is shown for model B-aa, in which the seepage entrance was at a vertical face 400 ft from the line of wells, on figure 27. Equipotential lines as observed in model B-a are shown for a well spacing of 29 ft and well penetrations of 100% and 25% on figure 30.



MODELS B-a, B-b, AND B-c
 HYDROSTATIC PRESSURES BENEATH TOP STRATUM
 SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS
 AND OPEN RIVERSIDE BORROW PIT
 WELL SPACING= 29 FT

Figure 25



MODEL B-a

WELL FLOWS AND LANDSIDE SUBSTRATUM PRESSURES

SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS

IMPERVIOUS LANDSIDE TOP STRATUM

Figure 26

63. The test data presented on figures 25 and 26 show that in order to achieve a high degree of pressure relief in stratified foundations the well screens must penetrate into the more pervious of the various strata. For example, wells fully penetrating the coarse sand stratum on 100-ft centers gave a pressure reduction of 95% H, whereas wells on the same spacing but only penetrating the top 25 ft of fine sand reduced the maximum landward pressure only 58% (see fig. 26). A comparison of the effectiveness of partially penetrating wells in a homogeneous foundation and a stratified foundation as in model B may be obtained from figures 15 and 26. The top stratum and distance to the source of seepage were the same for models A-a-1 and B-a. Although the average horizontal permeability in model B was the same as the permeability of the foundation in model A the well flow for 100% penetration was only about two-thirds that for the same seepage entrance and top strata in model A-a-1 (see figs. 15 and 26). This is because the depth of the pervious foundation in model B was only two-thirds that of model A. Although the case for 75% well penetration was not run in model B, it is pointed out that a 75% well penetration in this model would have resulted in 50% penetration of the coarse sand stratum and therefore would have given almost as efficient pressure relief as the 100% penetration. Fifty per cent penetration of the principal water-carrying stratum usually results in effective pressure relief (see fig. 15).

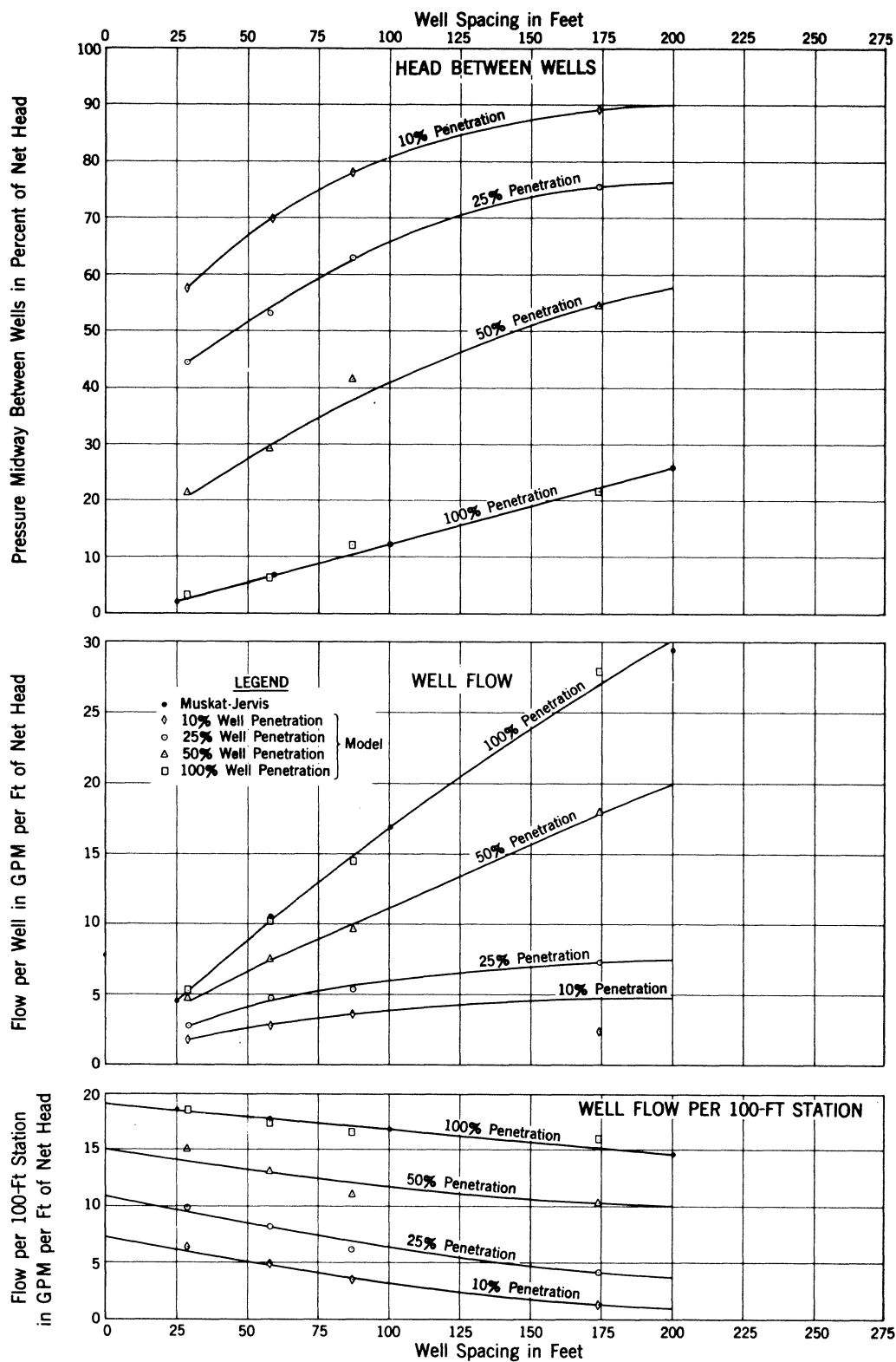
64. It appears from the data presented on figure 26 that a well penetration of approximately 75% would be required for efficient pressure relief for conditions as represented by model B-a. It may also be noted from figure 26 that close agreement was obtained between the model results

and the values computed from the Muskat-Jervis design curves for the case of 100% well penetration.

65. Model B-aa was identical to model B-a except that the source of seepage was 400 ft from the line of wells rather than 1000 ft as in model B-a. The results of the tests made in model B-aa are shown on figure 27. Because of the closer seepage entrance, closer well spacings were required in this model to achieve the same pressure reduction as obtained in model B-a. As was expected, because of the shorter seepage path the well flows for the same well spacing and penetration were approximately 2-1/2 times those obtained in model B-a. As observed in model B-a penetration of the wells into the principal water-carrying stratum was required in order to achieve effective pressure relief. Well flows and pressures midway between wells were also computed for model B-aa from the Muskat-Jervis curves for 100% well penetration.

Seepage entrance -- open riverside
borrow pits, model B-b

66. In this model the only seepage entrance into the pervious foundation was in a 100-ft borrow pit at the riverside toe of the levee which was excavated to the top of the sand; the vertical seepage entrance face 1000 ft from the line of wells used in other models was closed. Otherwise the model was the same as model B-a. Well flows and maximum residual landside pressures obtained in this model are shown on figure 28. Hydrostatic pressures immediately beneath the top stratum are shown for a well spacing of 29 ft and 0 ft (open vertical trench) and various penetrations on figure 25. Equipotential and flow lines are shown for model B-b on figure 30 (center sketch).



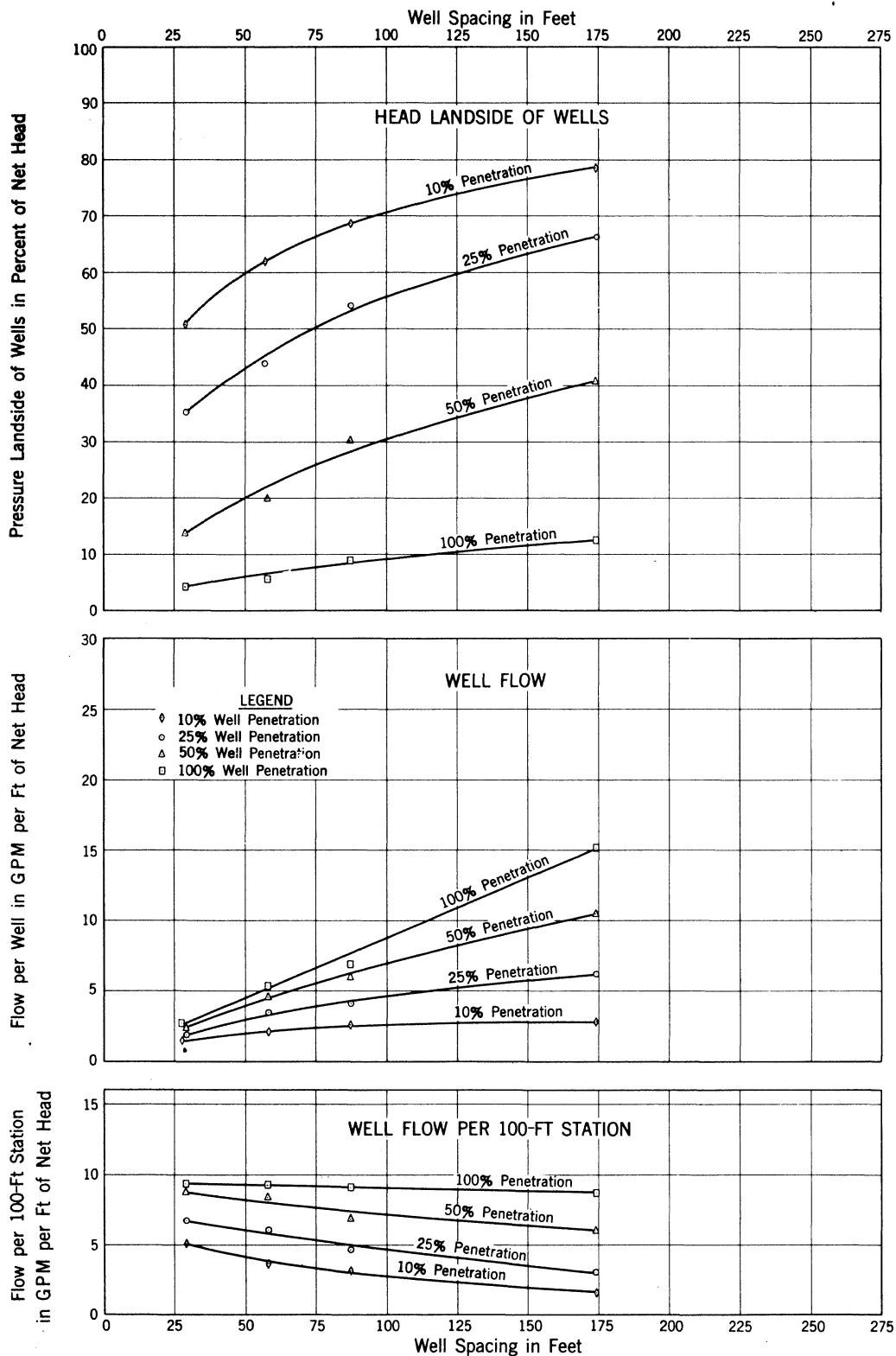
MODEL B -aa

WELL FLOWS AND LANDSIDE SUBSTRATUM PRESSURES

SEEPAGE ENTRANCE AT RIVER 400 FT FROM WELLS

IMPERVIOUS LANDSIDE TOP STRATUM

Figure 27



MODEL B-b
WELL FLOWS AND LANDSIDE
SUBSTRATUM PRESSURES
 SEEPAGE ENTRANCE 100-FT BORROW PIT 300 FT FROM WELLS
 IMPERVIOUS LANDSIDE TOP STRATUM

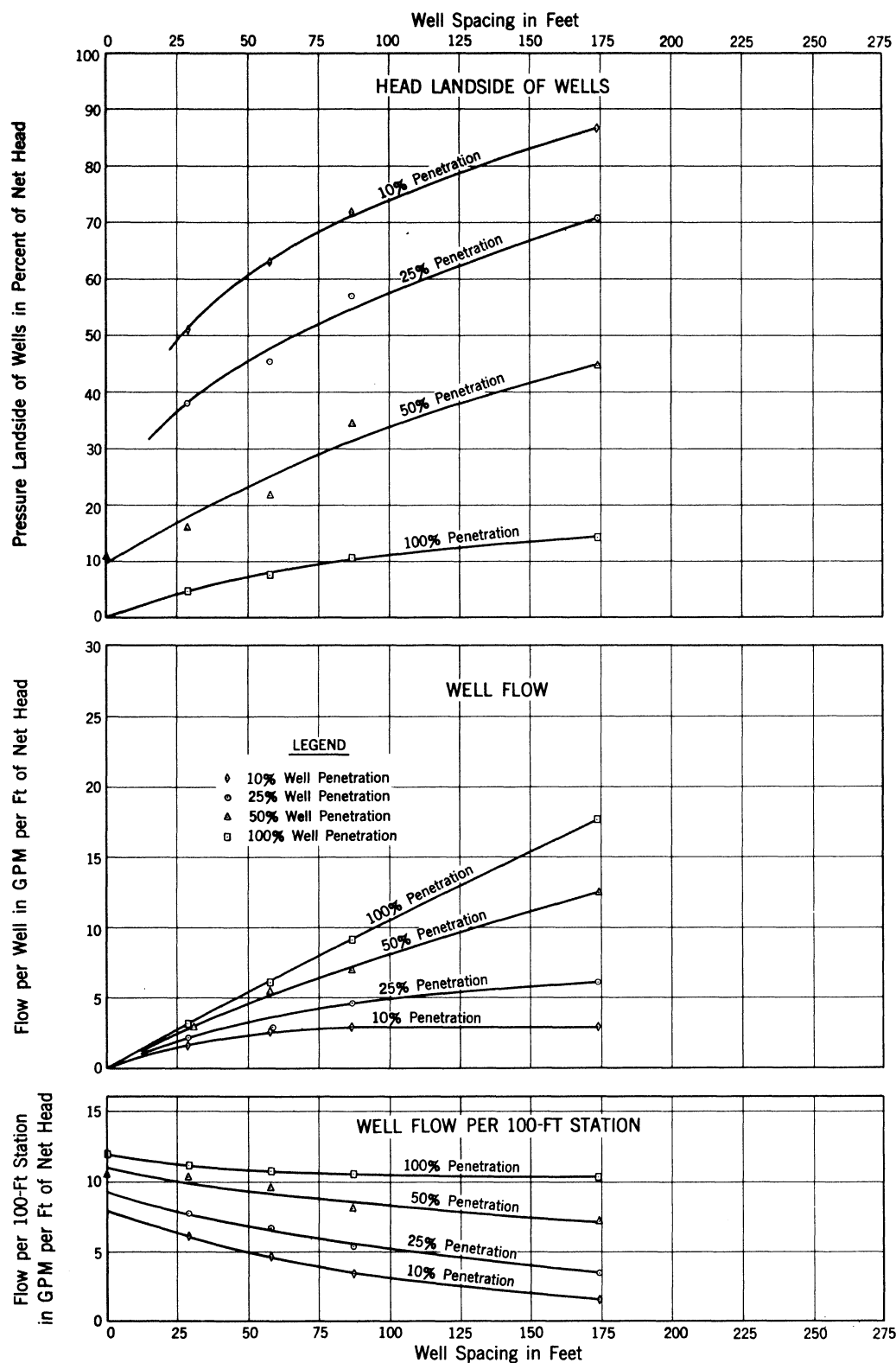
Figure 28

67. It may be noted from the data presented on figure 28 that the 100-ft open borrow pit, located riverside of the levee as shown in figure 7, with the river at an infinite distance away permitted more seepage (approximately 15%) to enter the pervious foundation, for the same drainage facilities, than entered the foundation in model B-a with the river 1000 ft from the wells (see figs. 26 and 28). In order to obtain the same landward pressure reduction closer well spacings were required in this model than were required in model B-a. The open riverside borrow pit as shown in figure 7 for model B-b was equivalent in seepage flow characteristics to a vertical seepage entrance face located 725 ft from the line of drainage wells. This distance was computed graphically by projecting riverward the straight-line portion of the hydraulic gradient beneath the levee (see fig. 25).

Seepage entrance at river 1000 ft
and in open riverside borrow pit, model B-c

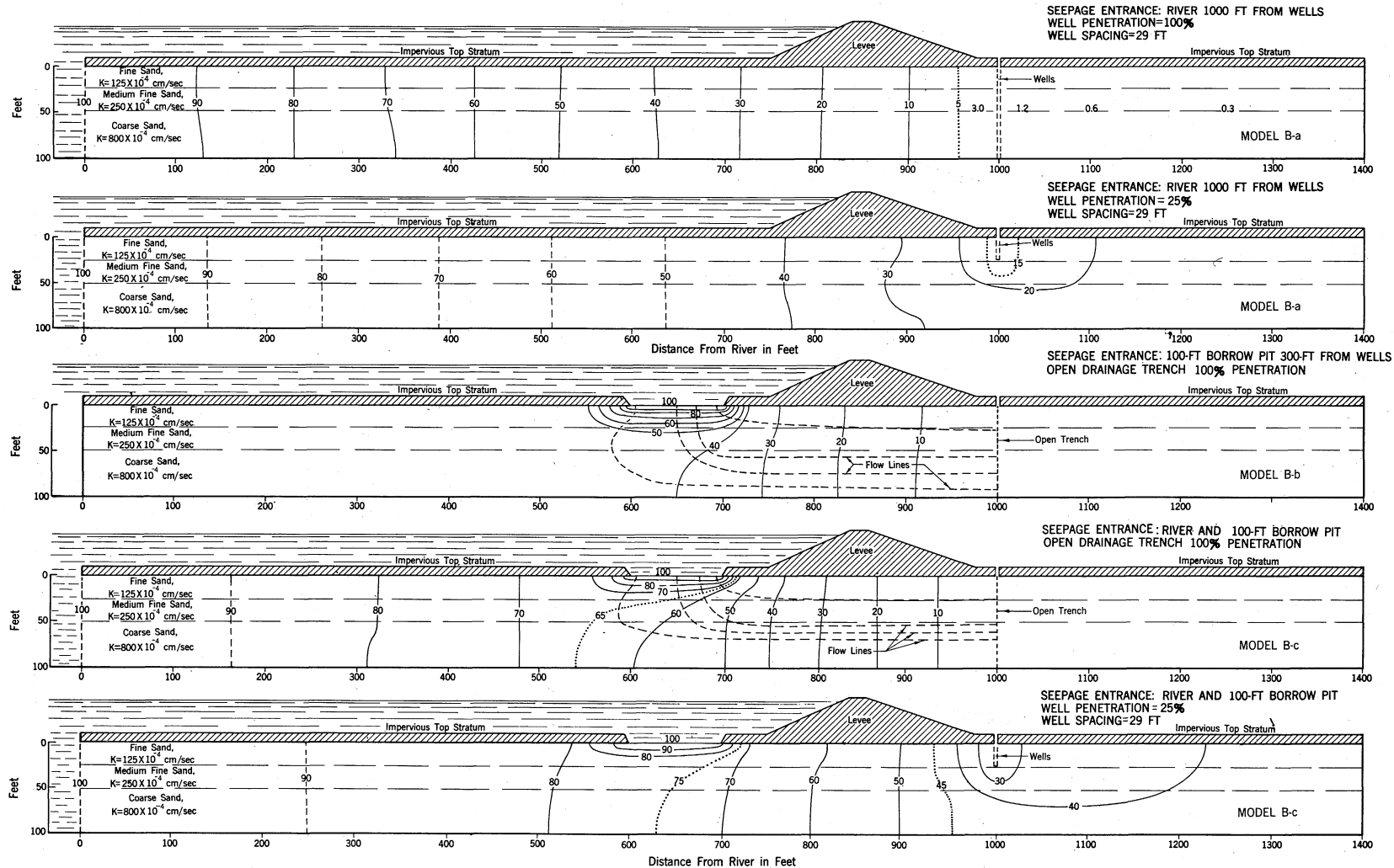
68. In this model the seepage could enter the pervious foundation at a vertical entrance face 1000 ft from the line of wells and also through the open borrow pit previously described (see fig. 7, model B-c). The hydrostatic pressures obtained in this model immediately beneath the impervious top stratum are shown for a well spacing of 29 ft and various well penetrations in the lower portion of figure 25. Well flows and land-side pressures for various well spacings and penetrations are shown for this model on figure 29. Equipotential and flow lines obtained for well spacings of 29 and 0 ft (open vertical trench) for penetrations of 25% and 100%, respectively, are shown in the lower two sketches of figure 30.

69. Where the seepage entered the foundation at both the river and



MODEL B-c
**WELL FLOWS AND LANDSIDE
 SUBSTRATUM PRESSURES**
 SEEPAGE ENTRANCE AT RIVER AND 100-FT BORROW PIT
 IMPERVIOUS LANDSIDE TOP STRATUM

Figure 29



MODELS B-a, B-b, AND B-c
EQUIPOTENTIAL AND FLOW LINES WITH RELIEF WELLS
SEEPAGE ENTRANCE AT RIVER 1000 FT FROM WELLS AND OPEN RIVERSIDE BORROW PITS

Figure 30

borrow pit the well flows, for 50% and 100% penetration, were increased approximately 50% over those obtained where the seepage could enter the foundation at the river only (compare figs. 26 and 29). It may also be noted from these figures that a closer well spacing was necessary for this model than for either model B-a or B-b in order to achieve the same land-side pressure reduction.

Models B-a, B-b, and B-c

70. Summarizing the results obtained in model B, it may be stated that where the foundation is stratified as in this model, the wells must penetrate into the principal water-carrying strata to effectively reduce landside pressures (see figs. 25-30). The best comparison of hydrostatic pressures beneath the top stratum as obtained for various seepage entrances in this model is shown by the hydraulic grade lines of figure 25. The effect of open riverside borrow pits on the seepage-flow pattern in the foundation simulated by model B is illustrated by figure 30.

Model C -- Greenville Levee Model

71. Throughout the well tests the development of sand boils landward of the levee was carefully avoided by keeping the river at such a stage that the substratum pressures did not reach the critical value required for the creation of sand boils. During these tests the top stratum was practically impervious. After the data on operation of the wells were obtained, the wells were closed and the river stage raised to the top of the levee. The excess pressure created caused a certain amount of heaving and loosening of the compacted soil that resulted in a considerable

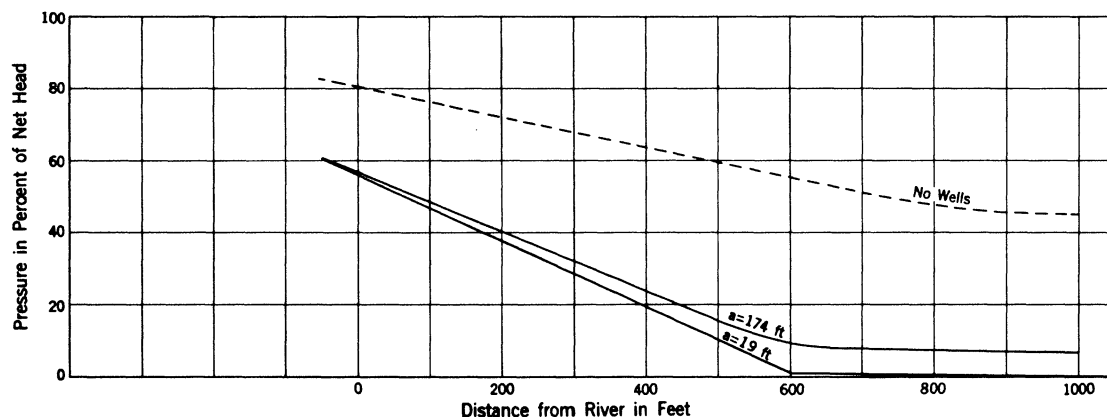
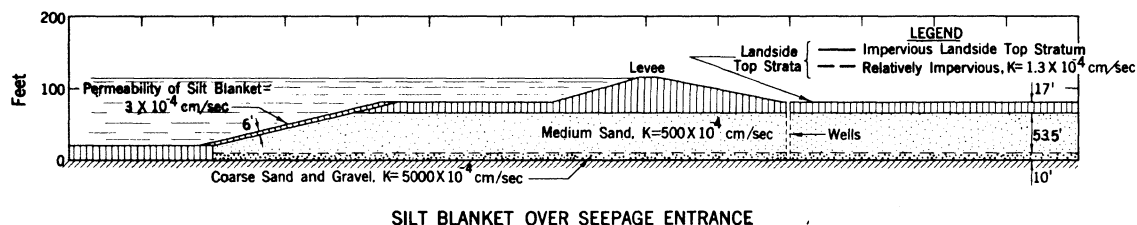
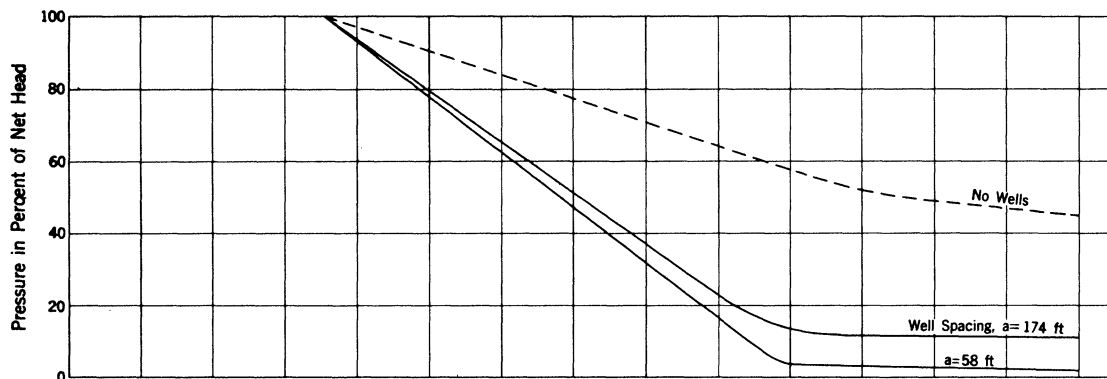
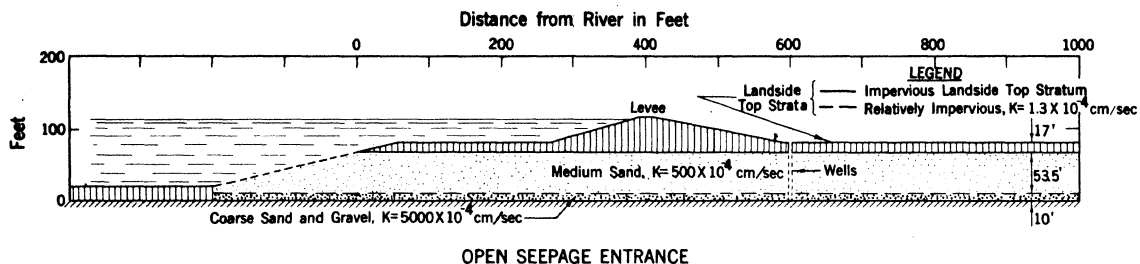
increase of perviousness as compared to its original condition. Because of the uniformity in thickness and characteristics of the top stratum no active sand boils developed; however, a considerable quantity of seepage occurred through the blanket which resulted in a lowering of the substratum pressures to a value approximately equal to the critical pressure required for sand boils (see fig. 31). A summary of all the test data and theoretical analyses for this model is given in table 4.

Model C-a

72. Hydrostatic pressures and well flows for model C-a are shown for fully penetrating wells at various spacings on figures 31 and 32. From the data shown on figure 32 it may be seen that for fully penetrating wells efficient pressure reduction was obtained for the well spacings tested. For 100% penetration and the same well spacing the relief wells in this model gave approximately the same pressure reduction landward of the levee as was observed in models A and B except for models A-b-1 and -2 and model B-aa.

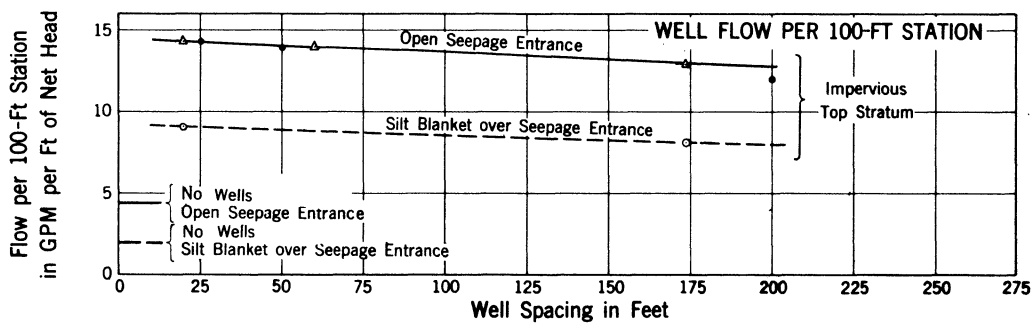
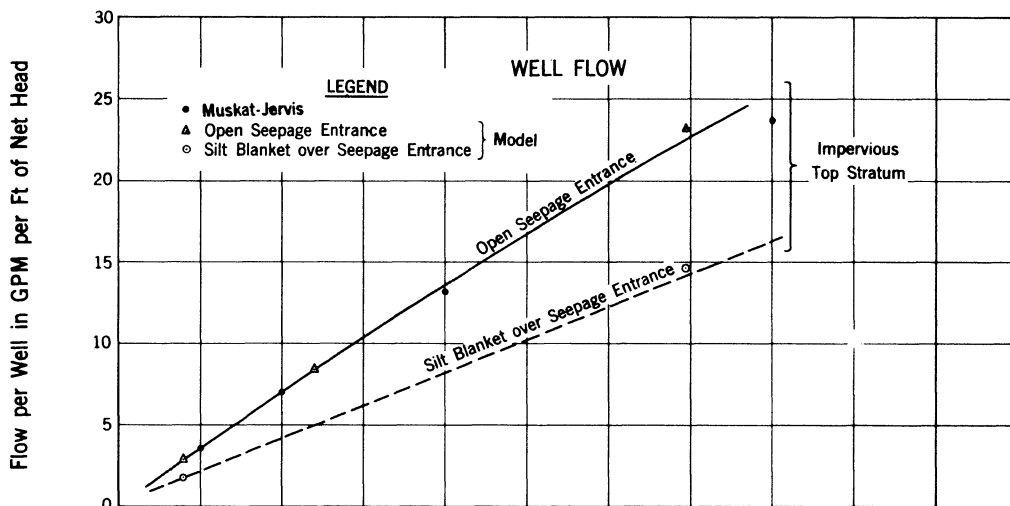
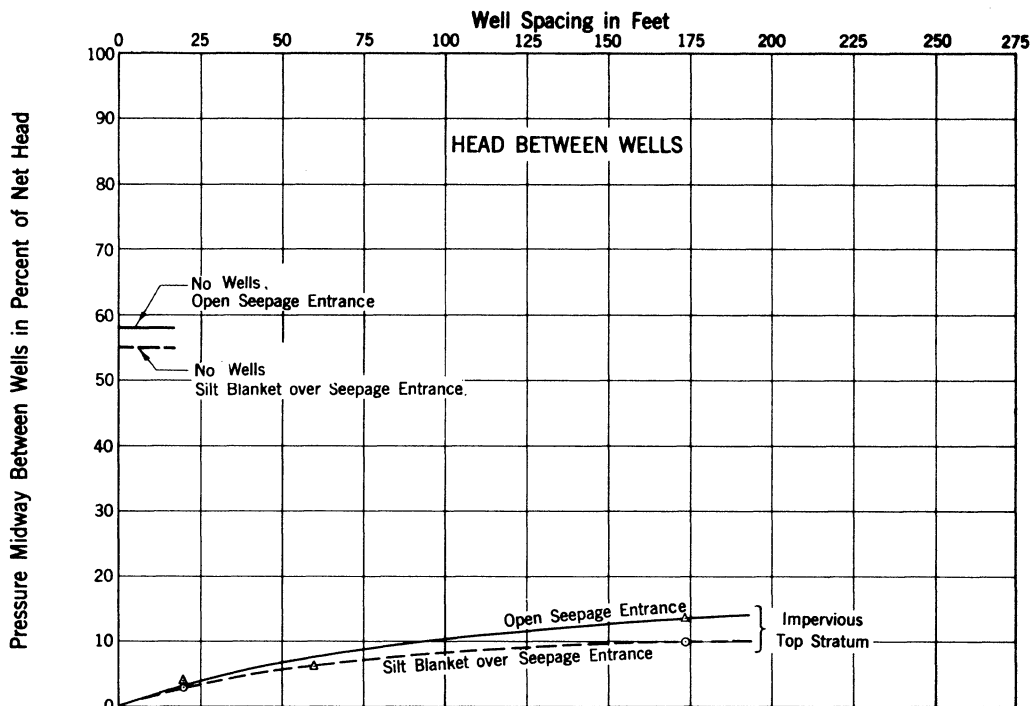
Model C-b

73. The silt blanket over the seepage entrance in the river channel had relatively little effect on the landward pressures with no wells in operation (see figs. 31 and 32). The silt blanket reduced the well flow by approximately one-third; however, it had relatively little effect on the residual pressures between the wells (see fig. 32). The silt blanket, in effect, increased the effective seepage path from the line of wells from about 650 ft to about 1100 ft. The results obtained in this model suggest that the placement of impervious blankets over exposed



MODELS C -a AND C -b
HYDROSTATIC PRESSURES BENEATH TOP STRATUM
SEEPAGE ENTRANCE AT RIVER 600 FT FROM WELLS
WELL PENETRATION 100%

Figure 31



MODELS C-a AND C-b

WELL FLOW, SEEPAGE, AND LANDSIDE SUBSTRATUM PRESSURES

SEEPAGE ENTRANCE AT RIVER 600 FT FROM WELLS

WELL PENETRATION 100%

Figure 32

pervious strata is not an efficient method of reducing landside pressures. The effect of the blanket would probably have been more pronounced if it had been more impervious and especially if it had been considerably more impervious than the landside blanket. Practical difficulties may prohibit the successful blanketing of a river slope but such difficulties would not apply to blanketing riverside borrow pits excavated to sand nor areas upstream from dams.

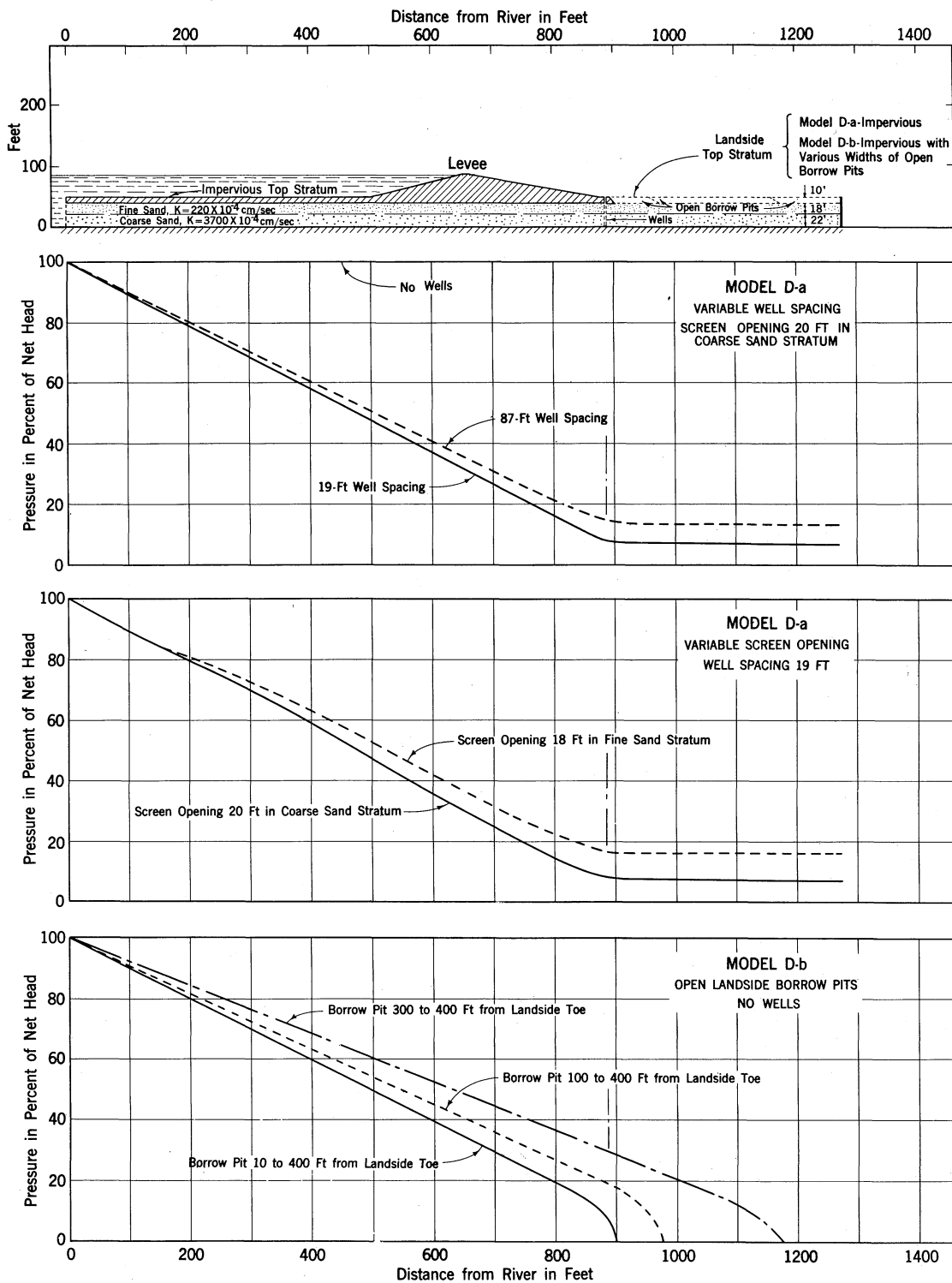
74. A more complete description of the results obtained in the Greenville levee model is contained in Waterways Experiment Station Technical Memorandum No. 182-1 "Seepage Model of Greenville Front Levee, Greenville, Mississippi."

Model D -- Memphis Levee Model

Model D-a

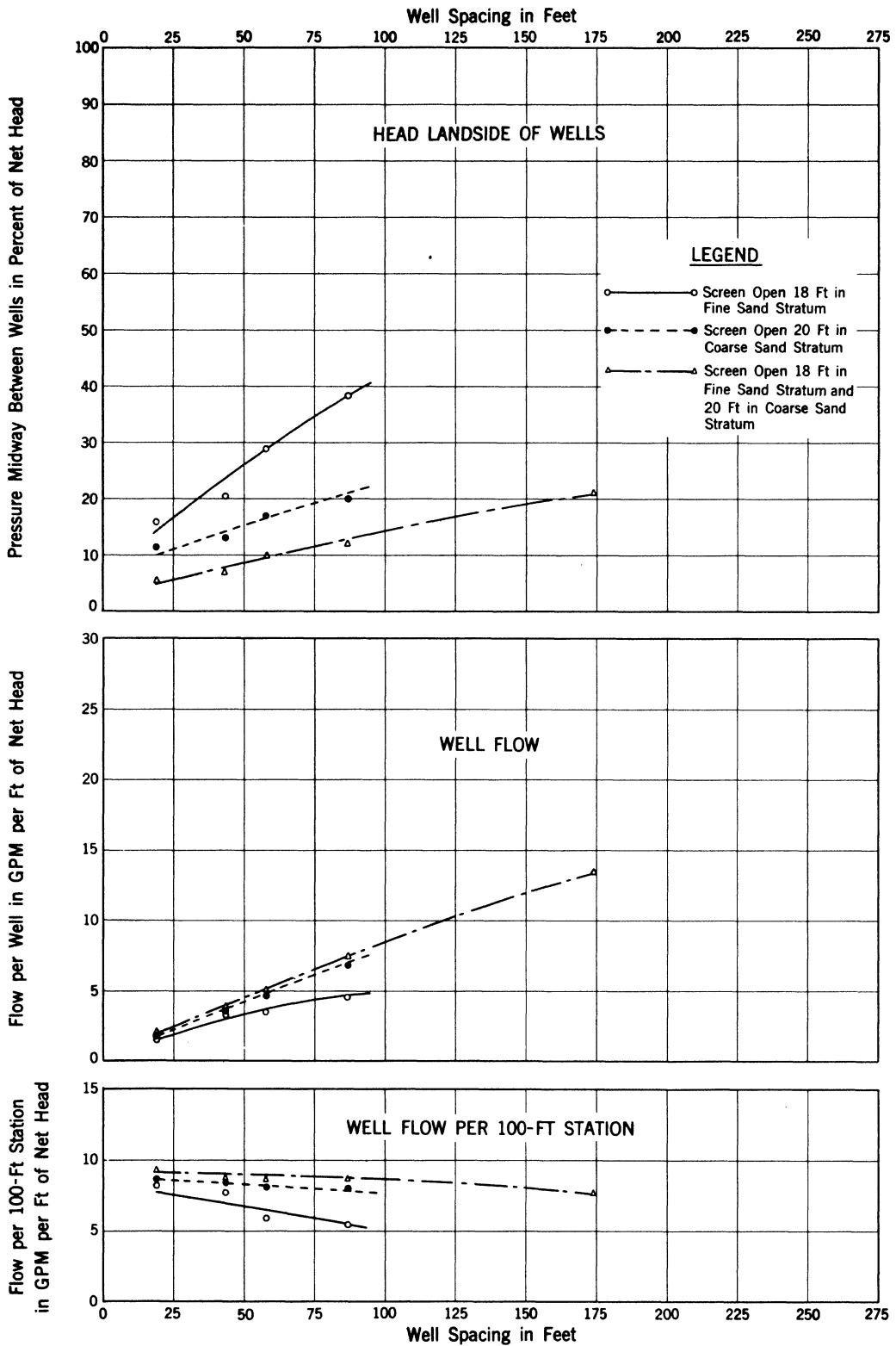
75. With no pressure relief facilities in this model the full hydrostatic pressure of the river was observed landward of the levee. With wells penetrating into the coarse sand stratum the landside pressure was reduced 85 to 90%, depending upon penetration of the well screen, for a 50-ft well spacing (see fig. 34). The hydraulic grade line from the river to landward of the levee is shown for various screen openings and penetrations in the upper two drawings of figure 33.

76. The importance of having the screen portion of relief wells penetrating into the principal water-carrying stratum is illustrated by the results presented in the upper portion of figure 34. For example, a well spacing of 100 ft and a screen penetration through only the upper stratum of fine sand reduced the hydrostatic pressure at the line of wells



MODELS D-a AND D-b
HYDROSTATIC PRESSURES BENEATH TOP STRATUM
SEEPAGE ENTRANCE AT RIVER 880 FT FROM WELLS

Figure 33



MODEL D-a
**WELL FLOW AND
 LANDSIDE SUBSTRATUM PRESSURES**
 SEEPAGE ENTRANCE AT RIVER 880 FT FROM WELLS
 VARIABLE WELL PENETRATION

by 58%, whereas wells with the screens penetrating the upper fine stratum plus 20 ft of the coarse stratum reduced the landside pressure 86% (see upper portion of fig. 34). Other test results obtained in this model but not plotted on figure 34 are given in table 5.

Model D-b

77. In this model landside drainage was obtained by opening up various widths of borrow pits landward of the levee. Tests were made in this model both with and without wells in operation. The hydraulic grade line beneath the top stratum without any wells in operation is shown for each of the three landside borrow pit conditions tested in the lower portion of figure 33. The borrow pits landward of the levee toe materially reduced the hydrostatic pressures at the toe of the levee, and were approximately as effective as a line of wells on 75-ft centers penetrating the coarse sand stratum (compare results shown on figs. 33 and 34). It is pointed out that for the same pressure reduction at the landside toe of the levee the flow from the landside borrow pits was almost exactly equal to that from the system of wells. Wells in combination with landside borrow pits were relatively little more effective in reducing landside pressures than were wells alone for the same spacing and penetration.

PART VII: CONCLUSIONS

78. The results of the model studies described in this report are the basis for the following conclusions:

- a. Relief wells with proper spacing and penetration will effectively reduce excess hydrostatic pressures landward of levees underlain by a pervious foundation for a wide range of seepage entrances, foundation stratification, and landward top strata.
- b. With adequate well spacing and penetration, seepage normally emerging landward of a levee (without wells) may also be materially reduced, although the total flow (well flow plus seepage with wells open) will be increased to some degree.
- c. For the models tested and test conditions applicable, there was close agreement between the observed well flows and landside pressures and the values computed from the various curves and formulas presented in the appendix.
- d. The pressures between wells and the well discharges are always slightly less for a relatively impervious landside blanket than for a completely impervious landside blanket, since the former contributes to pressure relief. Therefore, well flows and residual landside pressures for relatively impervious landside blankets will be less than those computed for an impervious landside top stratum.
- e. It is important that the wells penetrate into the principal water-carrying strata, in order to obtain efficient pressure relief where the pervious foundation is stratified (as in model B). The Muskat-Jervis curves for partial penetration are not applicable to this type of foundation. However, Muskat's curve for 100 per cent penetration is applicable if the average horizontal permeability is used for computing the well flow.
- f. Borrow pits excavated to sand on the riverside of a levee or dam increase underseepage and have a pronounced effect on the operation of relief well systems. In the model tested, landside borrow pits excavated to sand reduced the hydrostatic pressure at the toe of a levee and were approximately as effective as relief wells penetrating into the principal water-carrying strata. For the same pressure reduction at the landside toe of a levee, the seepage flow from the borrow pits was the same as from a line of relief wells.

- g. Although the tests reported in this memorandum covered certain specific foundation conditions, it is considered that the results obtained can, through use of good judgment, be extrapolated for use in designing well systems where conditions are generally similar.

TABLES

Table 1

OUTLINE OF TESTS

<u>Model</u>	<u>Seepage Entrance</u>	<u>Landside Top Stratum</u>	<u>Well Penetrations in Per cent</u>	<u>Well Spacings in Feet</u>	<u>Figure References</u>
A-a-1	River 1000 ft from wells	Impervious	25, 50, 100	23.6, 43.3, 86.6, 130, 260 ft, ∞	4, 6, 14, 15, 17
A-a-2	River 1000 ft from wells	Relatively impervious	0, 25, 50, 100	23.6, 43.3, 86.6, 130, 260 ft, ∞	4, 5, 6, 14, 16, 17, 18, 20
A-a-3	River 1000 ft from wells	No top stratum	0, 25, 50, 100	23.6, 43.3, 86.6, 130, 260 ft, ∞	4, 6, 14, 16, 17, 19
A-b-1	300 ft from wells	Impervious	50	23.6, 43.3, 86.6, 130, 260 ft ∞	4, 6, 21, 22
A-b-2	300 ft from wells	Relatively impervious	0, 50	23.6, 43.3, 86.6, 130, 260 ft, ∞	4, 5, 6, 20, 21, 22, 23
A-b-3	300 ft from wells	No top stratum	0, 50	23.6, 43.3, 86.6, 130, 260 ft, ∞	4, 6, 21, 22, 24
B-a	River 1000 ft from wells	Impervious	10, 25, 50, 100	29, 58, 87, 174 ft	7, 8, 25, 26, 30
B-aa	River 400 ft from wells	Impervious	10, 25, 50, 100	29, 58, 87, 174 ft	7, 8, 27
B-b	100-ft borrow pit only, 300 ft from wells	Impervious	10, 25, 50, 100	29, 58, 87, 174 ft	7, 8, 25, 28, 30
B-c	River plus borrow pit	Impervious	10, 25, 50, 100	29, 58, 87, 174 ft	7, 8, 25, 29, 30

Table 1 (Continued)

<u>Model</u>	<u>Seepage Entrance</u>	<u>Landside Top Stratum</u>	<u>Well Penetrations in Per cent</u>	<u>Well Spacings in Feet</u>	<u>Figure References</u>
C-a-1	Open river, 600 ft from wells	Impervious	100	19, 58, 174 ft	9, 10, 11, 31, 32
C-a-2	Open river, 600 ft from wells	Relatively impervious	-	No wells	9, 10, 11, 31, 32
C-b-1	River with silt blanket	Impervious	100	19, 58, 174 ft	9, 10, 11, 31, 32
C-b-2	River with silt blanket	Relatively impervious	-	No wells	9, 10, 11, 31, 32
D-a	River 880 ft from wells	Impervious $k=0.005 \times 10^{-4}$ cm/sec	Variable	19, 43.5, 58, 87, 174, ∞	11, 12, 13, 33, 34
D-b	River 880 ft from wells	Open Landside Borrow Pits	Variable	19, 43.5, 58, 87, 174, ∞	11, 12, 13, 33

Table 2

SUMMARY OF TEST DATA -- MODEL A

		Distance Seepage Entrance to Wells (ft)	Landside Top Stratum	Well Pene- tration (%)	Well Spacing (ft)	Head (%)		Reduction in Head due to Wells (%)	Well Flow (Q _w) (gpm)		Well Flow (Q ₁₀₀) (gpm)		Seepage (Q _s) (gpm)		Well Flow + Seepage Q ₁₀₀ + Q _s (gpm)	Reduction of Seepage due to Wells (%)	Increase of Total Flow due to Wells (%)	Ratio: Well Flow to Total Flow (%)
Model	Seepage Entrance					Model	Theo- retical	Model	Model	Theo- retical	Model	Theo- retical	Model	Theo- retical				
A-a-1	River	1000	Impervious	100%	23.6	1.1	0.6	98.9	2.8	2.6	12.0	11.2	0	-	12.0	0	0	-
					43.3	2.4	1.5	97.6	5.2	4.8	11.9	11.1	0	-	11.9	0	0	-
					86.6	5.0	3.9	95.0	10.3	9.5	11.9	10.9	0	-	11.9	0	0	-
					130	7.0	6.4	93.0	13.9	13.8	10.7	10.6	0	-	10.7	0	0	-
					260	12.2	14.4	87.8	26.6	25.8	10.2	9.9	0	-	10.2	0	0	-
A-a-1	River	1000	Impervious	50%	23.6	2.2	0.7	97.8	2.7	2.6	11.4	11.0	0	-	11.4	0	0	-
					43.3	3.5	2.3	96.5	4.8	4.7	11.1	10.8	0	-	11.1	0	0	-
					86.6	8.5	6.1	91.5	9.0	9.1	10.4	10.5	0	-	10.4	0	0	-
					130	10.8	10.4	89.2	13.1	13.1	10.1	10.1	0	-	10.1	0	0	-
					260	14.7	10.9	85.3	8.6	8.6	9.9	9.9	0	-	9.9	0	0	-
A-a-1	River	1000	Impervious	25%	23.6	6.5	1.6	93.5	2.7	2.5	11.4	10.6	0	-	11.4	0	0	-
					43.3	7.5	3.8	92.5	4.7	4.5	10.9	10.4	0	-	10.9	0	0	-
					86.6	14.7	10.9	85.3	8.6	8.6	9.9	9.9	0	-	9.9	0	0	-
					130	19.5	17.9	80.5	12.2	12.0	9.4	9.2	0	-	9.4	0	0	-
					260	34.6	38.6	63.0	19.3	19.5	7.4	7.5	0	-	7.4	0	0	-
A-a-2	River	1000	Relatively Impervious	100%	23.6	2.2	2.6	94.0	2.6	2.2	10.8	8.9	0.6	1.3	11.4	93	41	95
					43.3	2.7	2.9	92.5	4.6	3.9	10.6	9.0	0.7	1.4	11.3	91	41	93
					86.6	4.3	3.8	88.1	8.5	7.6	9.8	8.8	1.2	1.5	11.0	85	36	89
					130	6.3	5.5	82.5	11.8	11.0	9.1	8.5	1.7	1.7	10.8	79	34	84
					260	9.8	10.8	72.8	19.0	19.0	7.3	7.3	3.1	2.5	10.4	62	29	70
A-a-2	River	1000	Relatively Impervious	50%	23.6	2.6	-	92.8	2.4	-	10.0	-	0.6	-	10.6	93	32	94
					43.3	4.0	-	89.0	4.1	-	9.5	-	0.9	-	10.4	89	30	91
					86.6	7.2	-	80.0	7.5	-	8.6	-	1.7	-	10.3	79	28	84
					130	9.6	-	73.4	10.2	-	7.9	-	2.3	-	10.2	72	26	78
					260	18.0	-	50.0	16.6	-	6.4	-	3.7	-	10.1	54	24	64
A-a-2	River	1000	Relatively Impervious	25%	23.6	4.7	-	87.0	2.2	-	9.2	-	1.3	-	10.5	84	31	87
					43.3	6.6	-	81.7	3.8	-	8.7	-	1.7	-	10.4	79	30	83
					86.6	10.2	-	71.7	6.4	-	7.4	-	2.6	-	10.0	68	23	74
					130	14.3	-	60.3	7.9	-	6.1	-	3.4	-	9.5	58	18	64
					260	20.6	-	42.8	10.9	-	4.2	-	4.4	-	8.6	46	7	49
A-a-2	River	1000	Relatively Impervious	No Wells	No wells	36.0	-	-	-	-	-	-	8.1	7.2	-	--	--	--
A-a-3	River	1000	No Top Stratum	100%	23.6	0	-	-	1.9	-	8.1	-	2.2	-	10.3	78	4	79
					43.3	0	-	-	3.2	-	7.3	-	3.6	-	10.9	64	10	67
					86.6	0	-	-	5.0	-	5.7	-	4.9	-	10.6	51	7	54
					130	0	-	-	6.1	-	4.7	-	5.8	-	10.5	41	7	45
					260	0	-	-	7.0	-	2.7	-	7.6	-	10.3	23	4	26
A-a-3	River	1000	No Top Stratum	50%	23.6	0	-	-	1.5	-	6.3	-	4.1	-	10.4	58	5	60
					43.3	0	-	-	2.3	-	5.4	-	4.7	-	10.1	52	3	53
					86.6	0	-	-	3.5	-	4.1	-	6.0	-	10.1	39	2	40
					130	0	-	-	3.6	-	2.7	-	7.3	-	10.0	26	1	27
					260	0	-	-	3.5	-	1.3	-	8.6	-	9.9	13	1	13
A-a-3	River	1000	No Top Stratum	25%	23.6	0	-	-	1.1	-	4.5	-	5.3	-	9.8	47	0	46
					43.3	0	-	-	1.6	-	3.8	-	6.2	-	10.0	37	1	38
					86.6	0	-	-	1.7	-	2.0	-	7.8	-	9.8	21	-1	20
					130	0	-	-	1.9	-	1.5	-	8.2	-	9.7	17	-1	15
					260	0	-	-	1.9	-	0.7	-	9.2	-	9.9	6	1	7
A-a-3	River	1000	No Top Stratum	No Wells	No Wells	-	-	-	-	-	-	9.8	10.5	-	--	--	--	
A-b-1	300 ft from wells (Effective seepage distance to wells ~ 380')	380	Impervious	50%	23.6	9.6	2.0	90.4	6.1	6.6	25.9	28.0	0	-	-	0	0	--
					43.3	12.9	5.6	87.1	10.7	11.8	24.6	27.2	0	-	-	0	0	--
					86.6	23.7	14.5	76.3	18.3	21.7	21.2	25.1	0	-	-	0	0	--
					130	32.4	23.1	67.6	23.8	29.6	18.3	22.8	0	-	-	0	0	--
					260	50.3	48.5	49.7	37.2	46.5	14.3	17.9	0	-	-	0	0	--

Table 2 (Continued)

Model	Seepage Entrance	Distance Seepage Entrance to Wells (ft)	Landside Top Stratum	Well Pene- tration (%)	Well Spacing (ft)	Head (%)		Reduction in Head due to Wells (%)	Well Flow (Q_w) (gpm)		Well Flow (Q_{100}) (gpm)		Seepage (Q_s) (gpm)		Well Flow + Seepage $Q_{100} + Q_s$ (gpm)	Reduction of Seepage due to Wells (%)	Increase of Total Flow due to Wells (%)	Ratio: Well Flow to Total Flow (%)
						Model	retical		Model	retical	Model	retical	Model	retical				
A-b-2	300 ft from wells (Effective seepage distance to wells ~ 380')	380	Relatively Impervious	50%	23.6	7.3	-	87.1	6.5	-	27.7	-	1.4	-	29.1	89	117	95
					43.3	10.8	-	81.0	10.9	-	25.2	-	2.3	-	27.5	84	105	92
					86.6	18.6	-	67.3	18.3	-	21.2	-	3.7	-	24.9	72	86	85
					130	26.6	-	53.3	21.3	-	16.5	-	4.5	-	21.0	66	57	79
					260	37.6	-	34.0	30.5	-	11.8	-	5.6	-	17.4	58	30	68
					No wells	No wells	-	-	-	-	-	-	13.4	12.7	-	-	-	-
A-b-3	300 ft from wells (Effective seepage distance to wells ~ 380')	380	No Top Stratum	50%	23.6	0	-	-	3.3	-	14.1	-	11.4	-	25.5	56	0	55
					43.3	0	-	-	4.4	-	10.3	-	13.8	-	24.1	47	0	43
					86.6	0	-	-	6.4	-	7.4	-	17.0	-	24.4	35	0	30
					130	0	-	-	8.0	-	6.1	-	18.6	-	24.7	28	0	25
					260	0	-	-	8.1	-	3.1	-	20.5	-	23.6	21	0	13
					No wells	No wells	-	-	-	-	-	-	26.0	26.0	-	-	-	-

Notes: % Well Penetration = penetration of well screen into the pervious stratum in per cent of total stratum thickness.

Head = maximum head landward of levee in per cent of net head on levee.

Q_w = flow per well in gpm per ft of net head on levee.

Q_{100} = well flow per 100 ft of levee in gpm per ft of net head on levee.

$Q_{100} + Q_s$ = well flow plus seepage per 100 ft of levee in gpm per ft of net head on levee.

Theoretical values computed from curves and formulas given in appendix.

Seepage (Q_s), as used in this report, is the flow emerging landward of the line of wells.

Table 3
SUMMARY OF TEST DATA -- MODEL B

Model	Seepage Entrance	Distance Seepage Entrance to Wells (ft)	Well Spacing (ft)	100% Well Penetration						50% Well Penetration			25% Well Penetration			10% Well Penetration		
				Head (%)		Well Flow Q_w (gpm)		Well Flow Q_{100} (gpm)		Head (%)	Well Flow Q_w (gpm)	Well Flow Q_{100} (gpm)	Head (%)	Well Flow Q_w (gpm)	Well Flow Q_{100} (gpm)	Head (%)	Well Flow Q_w (gpm)	Well Flow Q_{100} (gpm)
				Model	Theoretical	Model	Theoretical	Model	Theoretical	Model	Model	Model	Model	Model	Model	Model	Model	Model
B-a	River	1000	0	0	0	-	-	7.3	-	0	-	6.9	0	-	6.4	0	-	4.8
			29	1.6	1.0	2.1	2.2	7.2	7.5	10.6	1.9	6.5	22.3	1.7	5.7	38.6	1.3	3.5
			58	3.1	2.6	4.2	4.3	7.2	7.3	14.6	3.6	6.2	30.4	3.0	5.1	50.7	2.1	3.3
			87	5.0	4.4	6.2	6.3	7.2	7.3	22.7	4.7	5.4	41.1	3.7	4.2	59.9	2.4	2.8
			174	8.5	10.0	11.8	12.0	6.8	6.9	34.5	9.2	5.3	54.2	5.7	3.3	78.7	2.6	1.5
B-aa	River	400	0	-	0	-	-	-	-	0	-	15.1	0	-	10.9	0	-	7.4
			29	2.7	2.5	5.4	5.3	18.5	18.4	21.4	4.9	15.0	44.5	2.9	9.9	57.5	1.9	6.5
			58	6.5	6.3	10.2	10.3	17.6	17.8	29.2	7.6	13.1	53.1	4.8	8.2	69.8	2.9	5.0
			87	12.4	10.3	14.5	14.9	16.6	17.1	41.5	9.7	11.1	63.0	5.4	6.2	77.9	3.7	3.6
			174	21.6	22.2	27.9	26.5	16.1	15.2	54.4	17.9	10.3	75.5	7.3	4.2	88.9	2.4	1.4
B-b	100-ft Borrow Pit	∞	0	0	-	-	-	8.8	-	13.6	2.5	8.7	35.0	1.9	6.7	50.8	1.5	5.0
			29	4.4	-	2.7	-	9.3	-	19.9	4.6	8.4	43.7	3.5	6.0	61.8	2.1	3.7
			58	5.5	-	5.3	-	9.1	-	30.2	6.0	6.9	54.0	4.1	4.7	68.5	2.6	3.0
			87	8.9	-	7.9	-	9.1	-	40.8	10.5	6.0	66.3	6.2	3.1	78.6	2.9	1.6
			174	12.8	-	15.2	-	8.7	-	-	-	-	-	-	-	-	-	-
B-c	River and 100-ft Borrow Pit	1000	0	0	-	-	-	11.9	-	10.9	-	10.5	-	-	-	-	-	-
			29	4.7	-	3.2	-	11.0	-	15.9	3.0	10.3	38.1	2.3	7.8	51.2	1.8	6.0
			58	7.5	-	6.2	-	10.7	-	21.6	5.5	9.5	45.3	2.9	6.7	62.9	2.7	4.7
			87	10.7	-	9.2	-	10.6	-	34.4	7.1	8.1	57.0	4.7	5.4	71.6	3.0	3.4
			174	14.6	-	17.8	-	10.2	-	44.7	12.5	7.2	70.9	6.2	3.6	86.6	3.0	1.7

Notes: % well penetration = penetration of well screen into pervious stratum in per cent of total stratum thickness.
Head = maximum head landward of levee in per cent of net head on levee.
Reduction in head due to wells = 100% - head (%) landward of levee.
 Q_w = flow per well in gpm per ft of net head on levee.
 Q_{100} = well flow per 100 ft of levee in gpm per ft of net head on levee.

Table 4

SUMMARY OF TEST DATA - MODEL C

		Distance Seepage Entrance To Wells (ft)	Landside Top Stratum	Well Pene- tration (%)	Well Spacing (ft)	Head (%)	Well Flow Q_w (gpm)		Well Flow Q_{100} (gpm)		Seepage Q_s (gpm)
<u>Model</u>	<u>Seepage Entrance</u>					<u>Model</u>	<u>Model</u>	<u>Theo- retical</u>	<u>Model</u>	<u>Theo- retical</u>	<u>Model</u>
C-a	River (open)	600	Impervious	100	19	4.0	2.8	---	14.4	---	---
			Impervious		25	---	---	3.6	---	14.4	---
			Impervious		50	---	---	---	---	13.9	---
			Impervious		58	6.0	8.4	---	14.0	---	---
			Impervious		174	13.6	23.2	---	12.7	---	---
			Impervious		200	---	---	23.6	---	12.0	---
			Rel. Imp.	-	No Wells	58.0	---	---	---	---	4.4
C-b	River (6-ft silt blanket over seepage entrance)	600	Impervious	100	19	3.0	1.8	---	9.1	---	---
			Impervious		58	---	---	---	---	---	---
			Impervious		174	10.0	14.6	---	8.0	---	---
			Rel. Imp.	-	No Wells	55.0	---	---	---	---	2.0

Notes: % Well Penetration = penetration of well screen into pervious stratum in per cent of total stratum thickness.

Head = maximum head landward of levee in per cent of net head on levee.

Q_w = flow per well in gpm per ft of net head on levee.

Q_{100} = well flow per 100 ft of levee in gpm per ft of net head on levee.

Q_s = seepage flow per 100 ft of levee in gpm per ft of net head on levee.

Reduction in head due to wells = 100% - Head (%) landward of levee.

Well tests were completed before tests with no wells were performed.

Table 5

SUMMARY OF TEST DATA - MODEL D

Model	Seepage Entrance	Distance Seepage Entrance to Wells (ft)	Landside Top Stratum	Screen Opening	Well Spacing (ft)	Head (%)	Reduction in Head Due to Wells (%)	Well Flow Q_w (gpm)	Well Flow Q_{100} (gpm)	Well Flow + Seepage $Q_{100} + Q_s$ (gpm)
D-a-1	River	880	Impervious	18 ft in fine sand stratum	19	16.5	83.5	1.6	7.9	-
					43.5	20.5	79.5	3.2	7.5	-
					58	29.0	71.0	3.4	5.7	-
					87	38.5	61.5	4.5	5.2	-
D-a-2	River	880	Impervious	10 ft in coarse sand stratum	19	12.1	87.9	1.7	8.7	-
					43.5	14.5	85.5	3.6	8.3	-
					58	19.5	80.5	4.5	7.8	-
					87	23.0	77.0	6.6	7.6	-
D-a-3	River	880	Impervious	18 ft in fine sand stratum and 10 ft in coarse sand	19	8.3	91.7	1.7	8.8	-
					43.5	9.7	90.3	3.7	8.6	-
					58	14.1	85.9	4.8	8.2	-
					87	16.4	83.6	7.1	8.1	-
D-a-4	River	880	Impervious	20 ft in coarse sand stratum	19	10.9	89.1	1.7	8.8	-
					43.5	11.6	88.4	3.7	8.6	-
					58	15.4	84.6	4.7	8.1	-
					87	17.0	83.0	7.1	8.1	-
D-a-5	River	880	Impervious	18 ft in fine sand stratum and 20 ft in coarse sand stratum	19	5.5	94.5	1.8	9.2	-
					43.5	6.8	93.2	3.8	8.8	-
					58	9.8	90.2	5.0	8.7	-
					87	12.4	87.6	7.4	8.6	-
					174	20.6	79.4	13.3	7.7	-
D-b-1	River	880	100-ft open landside borrow pit 300 ft from landside toe	18 ft in fine sand stratum and 20 ft in coarse sand stratum	19	7.8	71.9	-	-	8.7
					43.5	9.7	65.0	-	-	8.6
					58	10.2	63.2	-	-	8.3
					87	11.1	60.0	-	-	8.3
					174	16.2	41.6	-	-	7.9
				No Wells	∞	27.8	0	-	-	6.3

See notes at end of table.

(Continued)

Table 5 (Continued)

SUMMARY OF TEST DATA - MODEL D

<u>Model</u>	<u>Seepage Entrance</u>	<u>Distance Seepage Entrance to Wells (ft)</u>	<u>Landside Top Stratum</u>	<u>Screen Opening</u>	<u>Well Spacing (ft)</u>	<u>Head (%)</u>	<u>Reduction in Head Due to Wells (%)</u>	<u>Well Flow Q_w (gpm)</u>	<u>Well Flow Q_{100} (gpm)</u>	<u>Well Flow + Seepage $Q_{100} + Q_s$ (gpm)</u>
D-b-2	River	880	300-ft open	18 ft in fine sand	19	8.1	54.5	-	-	8.7
			landside bor-	stratum and 20 ft	43.5	8.6	51.6	-	-	8.6
			row pit 100 ft	in coarse sand	58	9.5	46.6	-	-	8.4
			from landside	stratum	87	10.5	41.0	-	-	8.4
			toe		174	12.7	28.6	-	-	8.1
				No Wells	∞	17.8	0	-	-	7.3
D-b-3	River	880	390-ft open	No Wells	∞	12.5	0	-	-	8.1
			landside bor-							
			row pit 10 ft							
			from landside							
			toe							

Notes: Head = maximum head landward of levee in per cent of net head on levee.
Reduction in head due to wells = $100\% - (H)\%$ landward of levee.
 Q_w = flow per well in gpm per ft of net head on levee.
 Q_{100} = well flow per 100 ft of levee in gpm per ft of net head on levee.
 $Q_{100} + Q_s$ = well flow plus seepage per 100 ft of levee in gpm per ft of net head on levee.

APPENDIX

APPENDIX

SUMMARY OF FORMULAS FOR DESIGN OF RELIEF WELL SYSTEMS

Introduction

1. During the past few years a number of formulas have been developed for the design of well systems to control underseepage beneath dams and levees founded on pervious foundations. The first of these formulas was developed by Muskat¹ for the case of an infinite line of equispaced wells completely penetrating a homogeneous, semi-infinite, pervious stratum overlain by a uniform impervious top stratum and underlain by a horizontal impervious stratum. The seepage entrance was assumed as a vertical plane parallel to and at a given distance from the line of wells. Subsequently, an analysis for the case of partially penetrating wells for the same entrance and foundation conditions was accomplished by Jervis² by means of electrical models.

2. In nature, the assumption of an impervious top blanket is not always permissible as the top stratum on either the riverside or landside of a levee or dam may be relatively pervious. Leakage through such top strata definitely affects landward seepage and pressure and the design of a well system. Approximate methods for taking into account seepage through riverside and landside top strata in the design of well systems have been

¹ Morris Muskat, "The Flow of Homogeneous Fluids Through Porous Media" (New York: McGraw-Hill Book Co., Inc., 1937)

² W. H. Jervis, "Design of Relief Well Systems," published in report entitled "Conference on Control of Underseepage" (Vicksburg: Waterways Experiment Station, 1945)

developed by Bennett³ and Barron⁴ for certain simplified foundation conditions. It is pointed out that all the formulas developed by Barron and Bennett are based on the assumption of laminar flow through the top stratum.

3. Approximate methods have also been developed for evaluating the effect of riverside borrow pits on underseepage and landward pressures for certain generalized conditions.

4. The purpose of this appendix is to present a summary of the principal formulas which are presently available for designing drainage well systems, and the formulas which were used in making the theoretical analyses presented in the preceding report. Also included are graphs and curves necessary for solution of some of the various formulas. Examples of analyses of various foundation conditions and well systems, showing the necessary computations, are also included. All the formulas presented are for homogeneous pervious foundations unless otherwise stated.

Formulas

5. Variables which must be considered in formulas for the design of a drainage well system are:

- a. Distance from line of wells to source of seepage, s , in ft,

³ P. T. Bennett, "The Effect of Blankets on the Seepage through Pervious Foundations," Transactions, A.S.C.E., III (1946); and "Comments on the Design of Relief Wells," published in "Conference on Control of Underseepage" (Vicksburg: Waterways Experiment Station, 1945)

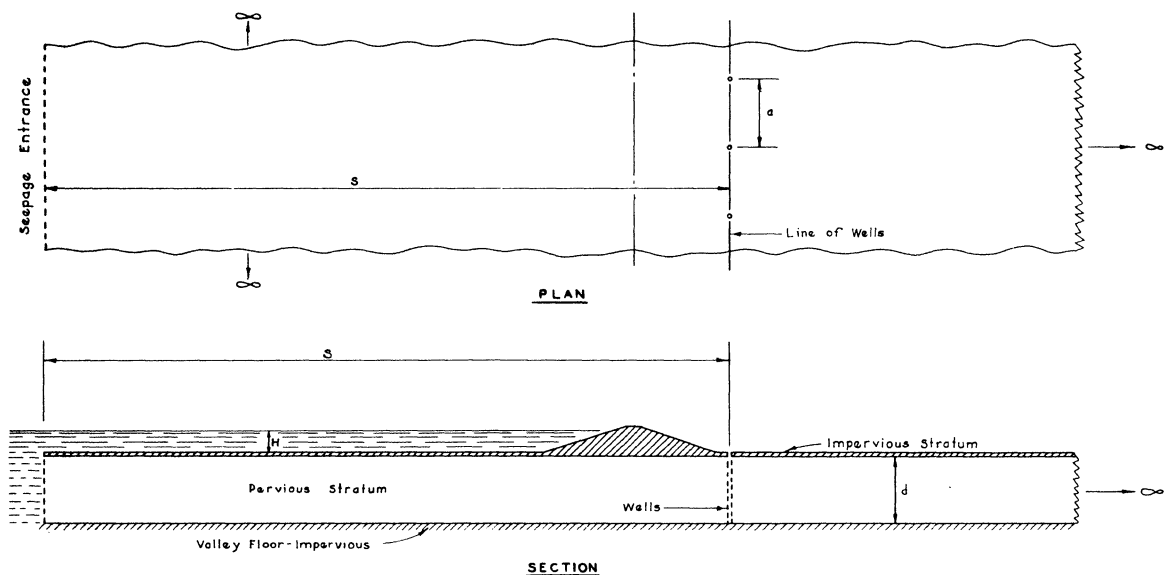
⁴ R. A. Barron, "The Effect of a Slightly Pervious Top Blanket on the Performance of Relief Wells," Proceedings-Second International Conference on Soil Mechanics, IV (1948)

- b. Depth of pervious layer, d , in ft,
- c. Coefficient of permeability of pervious foundation, k_f , in ft per min,
- d. Coefficient of permeability of top stratum, k_b , in ft per min,
- e. Spacing of wells, a , in ft,
- f. Penetration of wells into the pervious layer, W , in per cent,
- g. Difference in head between the source and discharge elevation, H , in ft,
- h. Effective radius of well, r_w , in ft,
- i. Length of foundation and top stratum landward of the line of wells, and
- j. Velocity and frictional head loss in the well screen and riser pipe.

Other features which influence the design and performance of a well system, but which do not lend themselves to theoretical analysis, are stratification of the foundation, lenticular deposits of silts and clays within the foundation, nonuniformity of the top stratum, and riverside or landside borrow pits. Because of these and other uncertainties that affect the design and performance of a well system, good judgement must be exercised in using the following theoretical formulas.

Case I -- Impervious Top Stratum
 Distance from Line of Wells to River = s ; Foundation Infinite
 in Extent Landward of Wells; Full and Partial Penetration of
 Pervious Stratum by Wells -- Muskat-Jervis

6. Well flows and pressures midway between wells may be computed for the case of an infinite line source, an infinite parallel line of wells, an impervious top stratum, and fully penetrating wells (see fig.



CASE I - IMPERVIOUS TOP STRATUM

Figure A1

A1) from the following formulas by Muskat:

$$P = H \left[1 + \frac{\log_e \frac{2}{1 + \cosh 2 j s}}{2 \log_e \frac{e}{j r_w}} \right] \dots \dots \dots (1)$$

where P = head midway between wells in ft, and

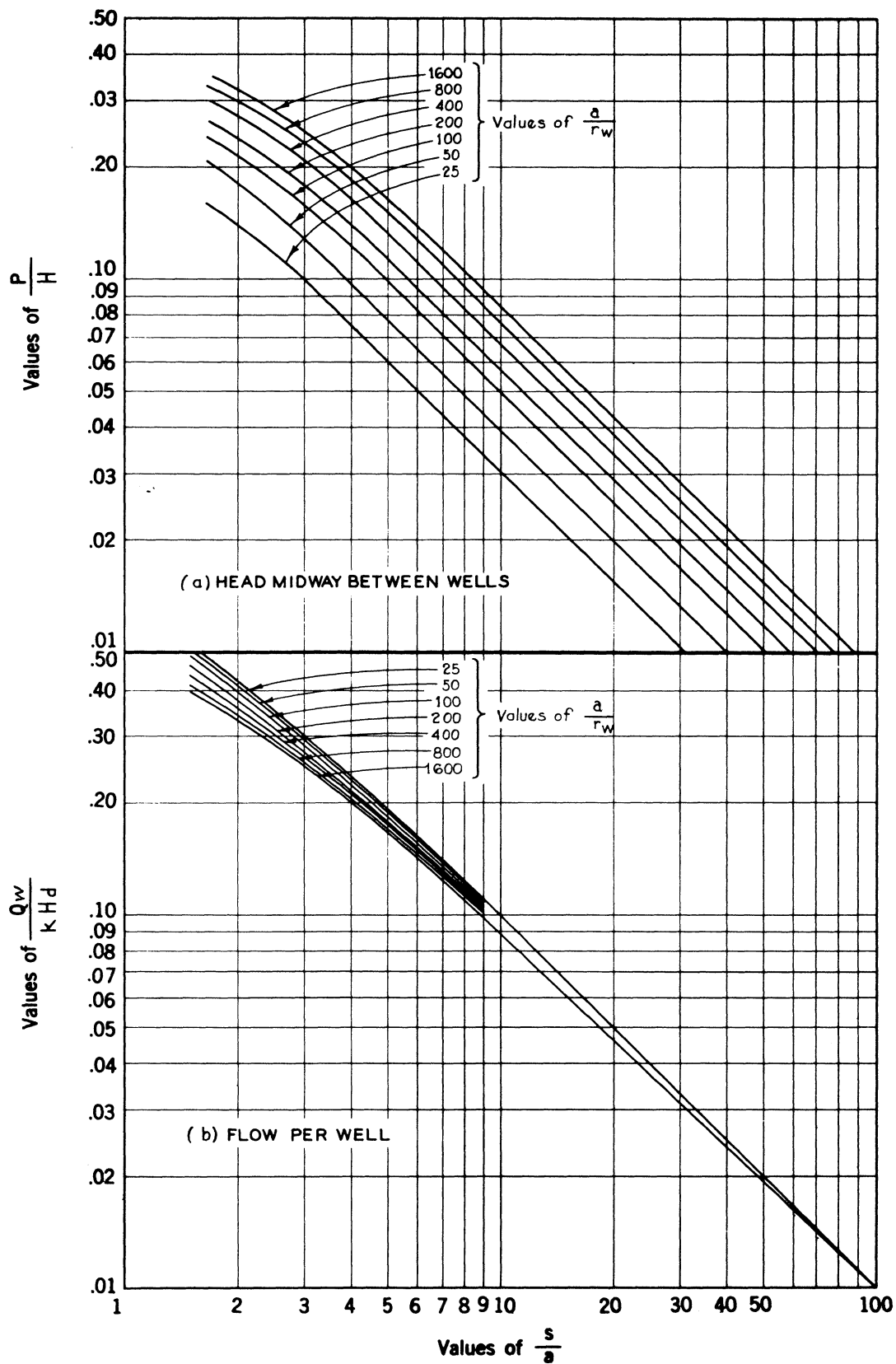
$$j = \frac{2\pi}{a}$$

$$Q_w = \frac{2\pi k H d}{\log_e \frac{e}{j r_w}} \quad \text{or} \quad \frac{k H d a}{s + \frac{a}{2\pi} \log_e \frac{a}{2\pi r_w}} \dots \dots \dots (2)$$

where Q_w = flow per well in cfm.

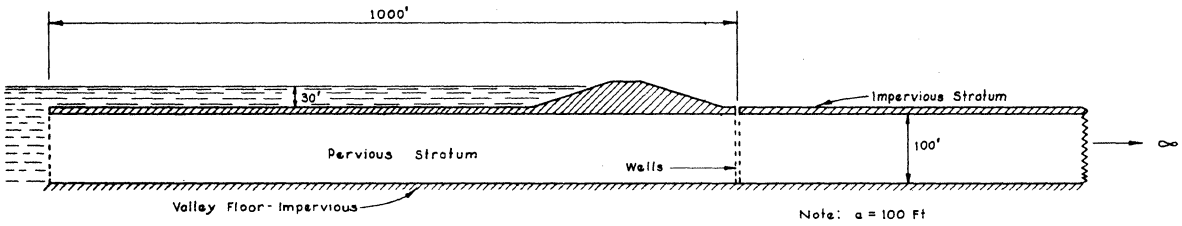
Charts for determining P and Q_w , based on the above formulas, are given in figure A2.

7. The following example shows the use of the above formulas and charts in solving a typical well problem (see fig. A3). For $r_w = 3$ in., $k = 500 \times 10^{-4}$ cm/sec = 0.10 ft per min, and $a = 100$ ft.



CHARTS FOR THE SOLUTION OF MUSKAT FORMULAS

Figure A2



CASE IMPERVIOUS TOP STRATUM-EXAMPLE

Figure A3

$$P = 30 \left[1 + \frac{\log_e \frac{2}{1 + \cosh 2 \frac{2\pi}{100} \times 1000}}{\frac{2\pi \times 1000}{2 \log_e \frac{e}{\frac{2\pi}{100} \times 0.25}}} \right] = 30 \left[1 + \frac{\log_e \frac{2}{1.924 \times 10^{+54}}}{2 \log_e \frac{1.922 \times 10^{27}}{\frac{\pi}{200}}} \right]$$

or $P = 1.97 \text{ ft}$ or $6.6\% H$,

from figure A2, $P = 0.067$ $H = 0.067 \times 30 = 2.01 \text{ ft} = 6.7\% H$ (assuming the head at the wells to be zero).

$$Q_w = \frac{2\pi \times 0.10 \times 30 \times 100}{2\pi \times 1000} = 27.2 \text{ cfm} = 204 \text{ gpm} = 6.80 \text{ H gpm}$$

from figure A2, $\frac{Q_w}{k H d} = 0.092$

or $Q_w = 0.092 \times 0.10 \times 30 \times 100 = 27.6 \text{ cfm} = 206 \text{ gpm} = 6.86 \text{ H gpm.}$

8. Formulas (1) and (2) are also applicable to stratified foundations where the wells completely penetrate the pervious strata and the coefficient of permeability used in formula (2) is the average horizontal permeability of the foundation.

9. Solution of this case for partially penetrating wells has been obtained by the Vicksburg District, CE, from electrical models. In these studies the carrying capacity of the pervious stratum was expressed, from

Darcy's law, as:

$$Q_M = \frac{k H a d}{s} \quad (3)$$

where Q_M = maximum possible discharge from pervious stratum in cfm,

a = well spacing, or extent of pervious stratum at right angles to direction of flow, in ft.

The quantity (Q_M) represents the amount of water which would pass through the pervious stratum with a vertical face at the source giving free inflow and a vertical drainage face at the exit giving free outflow. With a system of wells; there is a longer path of flow, resulting in more resistance, and hence a discharge smaller than Q_M . The increased resistance to flow into a line of wells as compared to an open discharge face was represented in the Vicksburg District studies as an "extra length" in the path of flow. Thus, the flow per well (from equation 3) may be written

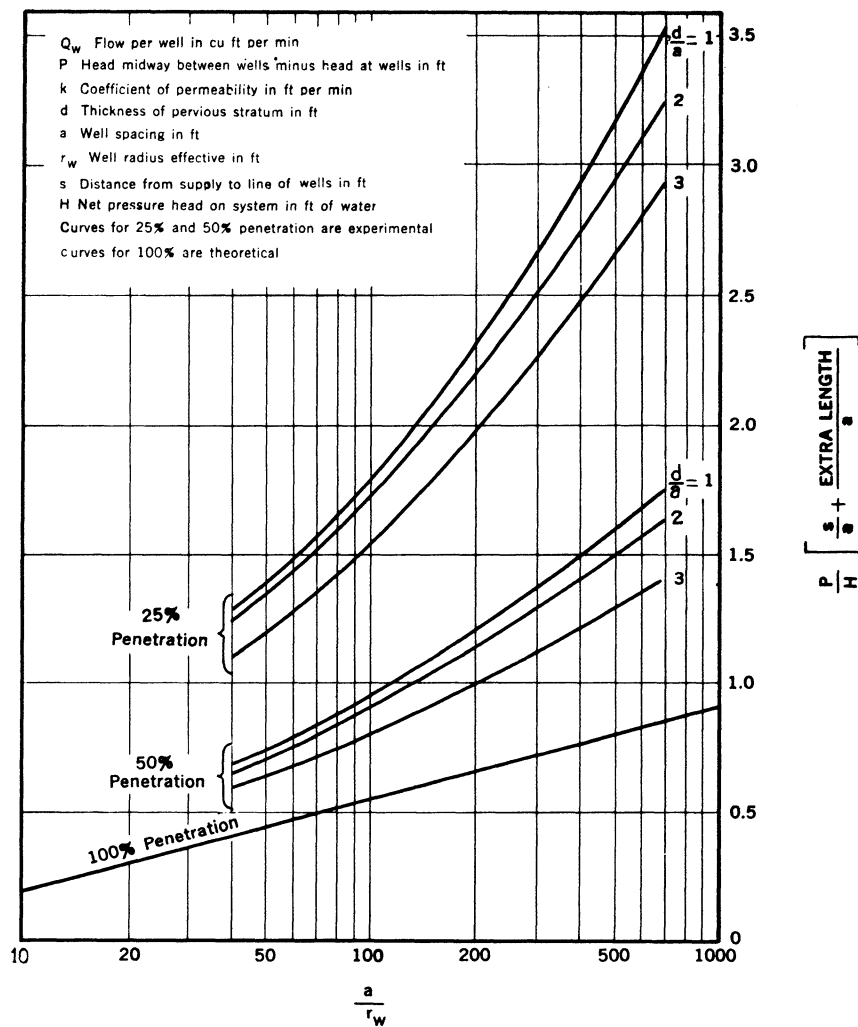
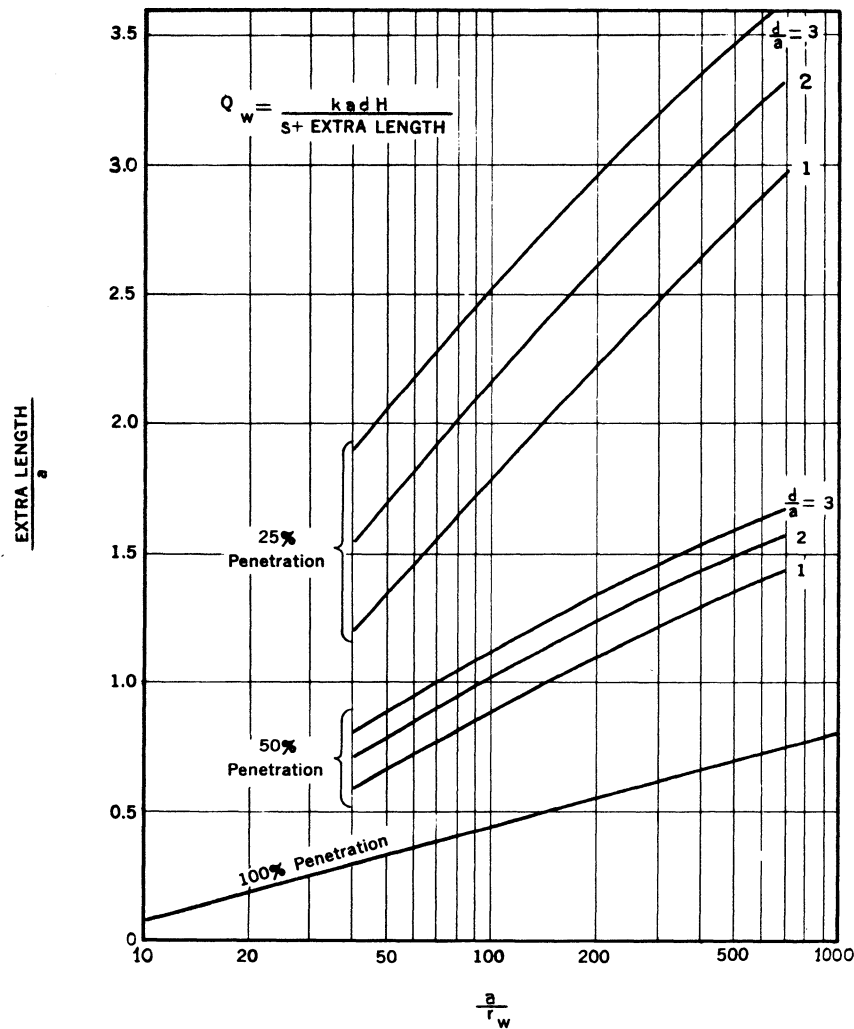
$$Q_w = \frac{k H a d}{s + \text{"extra length"}} \quad (4)$$

10. The value of the "extra length" factor in equation (4) may be obtained for both fully and partially penetrating wells from figure A4. A graph for determination of the head midway between partially penetrating wells is also included on figure A4.

11. The following example shows the use of the charts on figure A4 for solving a well problem with partially penetrating wells. The dimensions of the foundation and other factors are the same in this example as were used in the previous example except the wells only penetrate the upper 25 per cent of the pervious foundation.

12. In this example,

$$\frac{a}{r_w} = \frac{100}{0.25} = 400; \quad \frac{d}{a} = \frac{100}{100} = 1.0; \quad \text{and} \quad \frac{s}{a} = \frac{1000}{100} = 10.$$



ANALYSIS FOR RELIEF WELL SYSTEMS BY MUSKAT AND JERVIS

25% , 50% , AND 100% PENETRATION INTO PVIOUS STRATUM

(FROM FIGURE B "RELIEF WELLS FOR DAMS AND LEVEES" BY T. A. MIDDLEBROOKS AND WM. H. JERVIS, TRANSACTIONS, AM. SOC. CIVIL ENGINEERS, VOL. 112, 1947)

Figure A4

From figure A4, $\frac{\text{extra length}}{a} = 2.65$.

or extra length = $100 \times 2.65 = 265$ ft

$$Q_w = \frac{0.10 \times 30 \times 100 \times 100}{1000 + 265} = 23.7 \text{ cfm} = 177 \text{ gpm} = 5.90 \text{ H gpm}$$

Also from figure A4, $\frac{P}{H} \left[\frac{s}{a} + \frac{\text{extra length}}{a} \right] = 2.93$

$$\text{or } P = \frac{30 \times 2.93}{10 + 2.65} = 6.75 \text{ ft} = 23.2\% \text{ H.}$$

For an infinite number of wells (open drainage face) fully penetrating the pervious stratum, the flow would be 225 gpm per 100 ft of levee with zero head at the line of the wells.

14. In order to study the effect of well spacing, penetration, and radius on well flow and head between wells, several well systems have been analyzed in which the foundation conditions were maintained constant (see fig. A3) but the above factors were varied. The results of these analyses are shown in table A1. An examination of figure A4 and the data presented in table A1 indicated the following for homogeneous foundations and impervious top strata where the ratio $\frac{s}{a}$ is greater than 5.

- a. Well penetration (when > 25 per cent) and radius ($r_w > 1\text{-}1/2$ in.) have relatively little effect on well flow if the friction and velocity head losses in the well are negligible.
- b. Well penetration and spacing have a marked effect on the efficiency of the system in reducing landward pressures.
- c. Where the length of seepage path is long compared to the depth of the pervious stratum (i.e., greater than 10 to 1) the well flow per 100-ft station varies little for different well spacings.
- d. Where the length of seepage flow is only 2 to 5 times the depth of the pervious stratum, close well spacing and deep well penetration are needed to effectively reduce the head between wells.

Table A1TYPICAL RESULTS OF DESIGN COMPUTATIONS BASED ON
MUSKAT-JERVIS DESIGN CURVES

Screen Penetration W (%)	Well Diameter D (In.)	Well Spacing a (Ft)	Head Be- tween Wells P (%)	Well Flow Q_w (gpm)	Well Flow Q_{100} (gpm)
100	6	25	1.4	1.8	7.2
		50	3.2	3.6	7.2
		100	7.1	7.0	7.0
		200	14.9	12.9	6.4
100	24	25	0.8	1.9	7.6
		50	2.2	3.7	7.4
		100	5.3	7.2	7.2
		200	11.9	13.5	6.8
50	3	25	1.8	1.8	7.2
		50	6.5	3.5	7.0
		100	27.8	6.5	6.5
		200	----	---	---
50	6	25	1.6	1.8	7.2
		50	5.4	3.5	7.0
		100	13.3	6.6	6.6
		200	29.6	11.8	5.9
50	12	25	1.3	1.8	7.2
		50	4.3	3.6	7.2
		100	10.8	6.7	6.7
		200	24.8	12.1	6.0
50	24	25	1.2	1.8	7.2
		50	3.4	3.6	7.2
		100	8.6	6.9	6.9
		200	20.9	12.4	6.2
50	36	25	1.2	1.8	7.2
		50	3.0	3.6	7.2
		100	7.7	7.0	7.0
		200	17.5	12.7	6.4
25	6	25	2.5	1.7	6.8
		50	9.7	3.3	6.6
		100	23.2	5.9	5.9
		200	48.7	9.6	4.8
25	24	25	1.7	1.8	7.2
		50	6.2	3.4	6.8
		100	15.2	6.3	6.3
		200	27.1	10.7	5.4

See figure A3 for sketch of foundation conditions.

Head between wells, P, in per cent of net head on system.

Well flows (Q_w) based on $H = 1$ ft.

Friction and velocity head losses in well not considered in above computations.

15. For the same seepage entrance and pervious stratum, well flows computed from the Muskat-Jervis curves for an impervious top stratum are always greater than the well flows that would be obtained where the land-side top stratum is semi-pervious. Similarly, the head between wells as computed from the curves is always greater than that with a semi-pervious landside top stratum.

16. In designing a relief well system, cognizance must be taken of the hydraulic head losses in the collection, riser, and discharge pipes of the well, as such losses may materially affect its efficiency. The flow of water within a well requires a pressure head at the bottom sufficient to overcome the head losses within the well. As these head losses increase with increasing flow, the net head producing flow in the pervious stratum is reduced by an equivalent amount, thereby reducing the flow from the well. This process of adjustment continues until the flow through the foundation and from the well reaches a state of equilibrium so that the head loss in the foundation plus the head losses within the well equal the net head available. After computing the flow from a drainage well, the well discharge should be adjusted to this condition of equilibrium by considering the head losses within the well.

17. The method of adjusting the theoretically computed discharge to consider the head losses within the well so that $H = h_1 + h_2$ may be explained as follows and as shown by the example in figure A5: The total head loss (h_2) within the well is first computed (figs. A6 and A7) for several assumed flows, and plotted as on figure A5. The head loss (h_1) in the pervious stratum for various assumed flows is then plotted as on figure A5. Then the head loss curve for the well is added to the head loss curve

LEGEND

- Head loss in pervious stratum (h_1).
- - - - - Total head losses (friction + velocity + bend) in well (collection, riser, and discharge pipes).
- Head loss in pervious stratum plus head losses in well.

NOTES AND ASSUMPTIONS

Total head on levee, $H = 30$ ft of water.
 Computed flow through pervious stratum and well = 105 gpm assuming no head loss in well.
 Head loss in foundation = h_1 ft of water.
 For equilibrium of flow, $h_1 + h_2 = H =$ total head on levee.
 Length of well screen = 100 ft. (Equivalent length for computing friction loss = $1/2 \times 100 = 50$ ft.) $C = 100$.

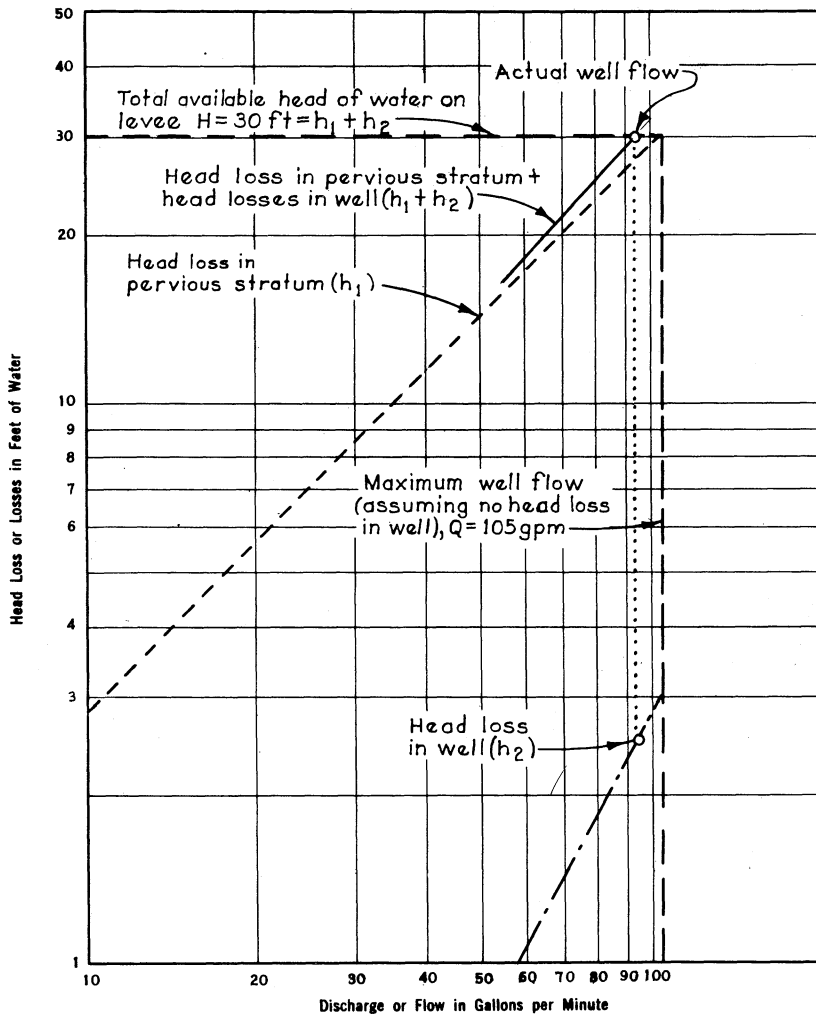
NOTES AND ASSUMPTIONS (CONTINUED)

Length of riser pipe = 10 ft, $C = 120$.
 Length of discharge pipe = 10 ft, $C = 120$.
 One standard 90° elbow connection between riser and discharge pipes.
 Velocity head loss, $h_v = V^2/2g$, ft of water.
 Loss in 90° elbow, $h_e = 1.5 V^2/2g$, ft of water.
 Hazen Williams pipe flow formula,

$$V = C R^{0.63} S^{0.54} \times 1.318, \text{ ft/sec.}$$

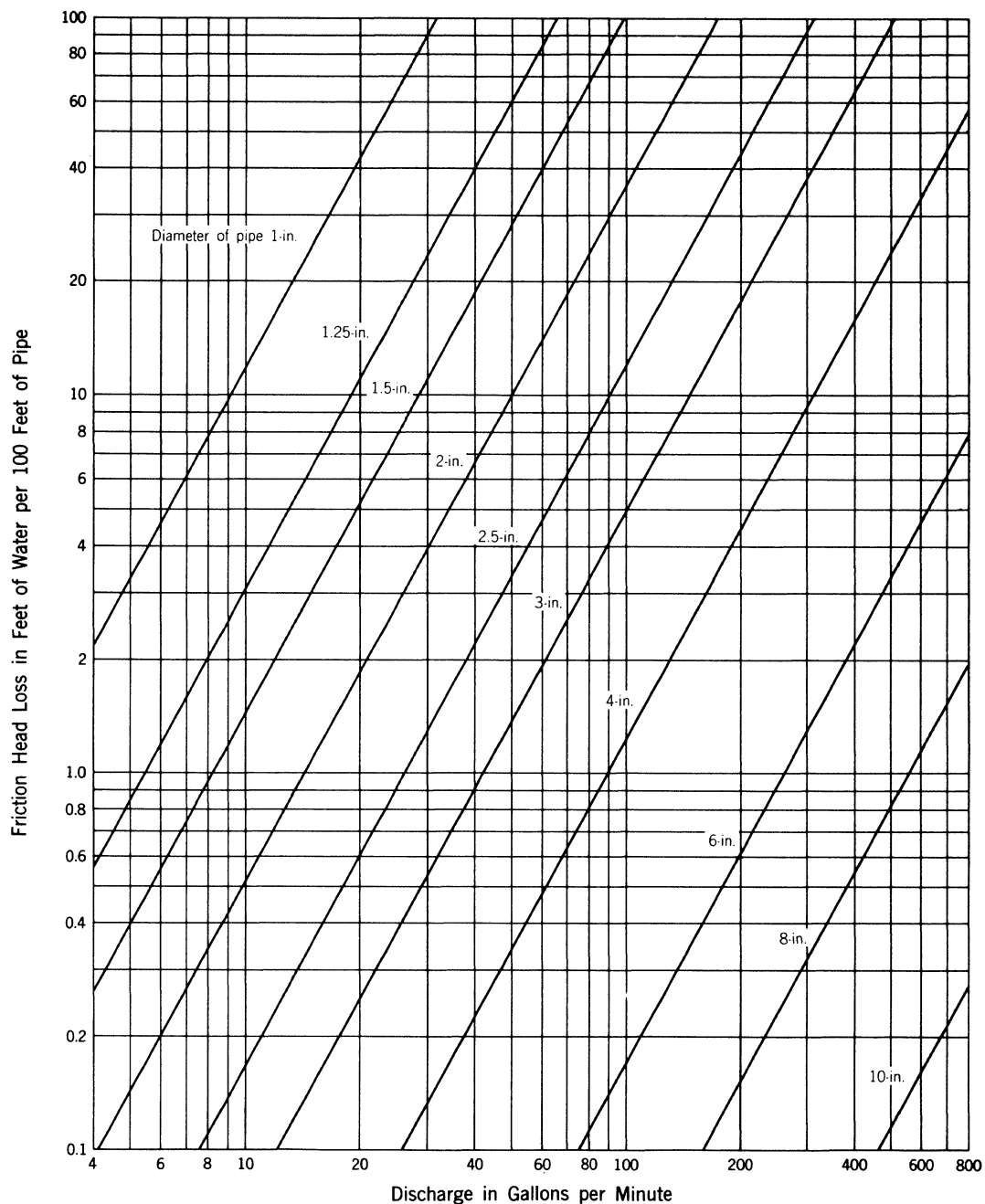
$$\text{or } h_f = \frac{303 V^{1.85}}{C^{1.85} d^{1.167}} \text{ ft/100 ft of pipe.}$$

Permeability of pervious stratum, $k = 0.1$ ft/min.
 Depth of pervious stratum, $d = 100$ ft.
 Well spacing, $a = 100$ ft.
 Distance from seepage entrance to line of wells, $s = 1000$ ft.
 Effective well radius, $r_w = 1.5$ in. = 0.125 ft.
 Penetration of pervious stratum by well screen = 100 per cent.



EXAMPLE OF GRAPHICAL SOLUTION TO DETERMINE THE REDUCTION OF COMPUTED FLOW BY HYDRAULIC HEAD LOSSES IN A RELIEF WELL

Figure A5

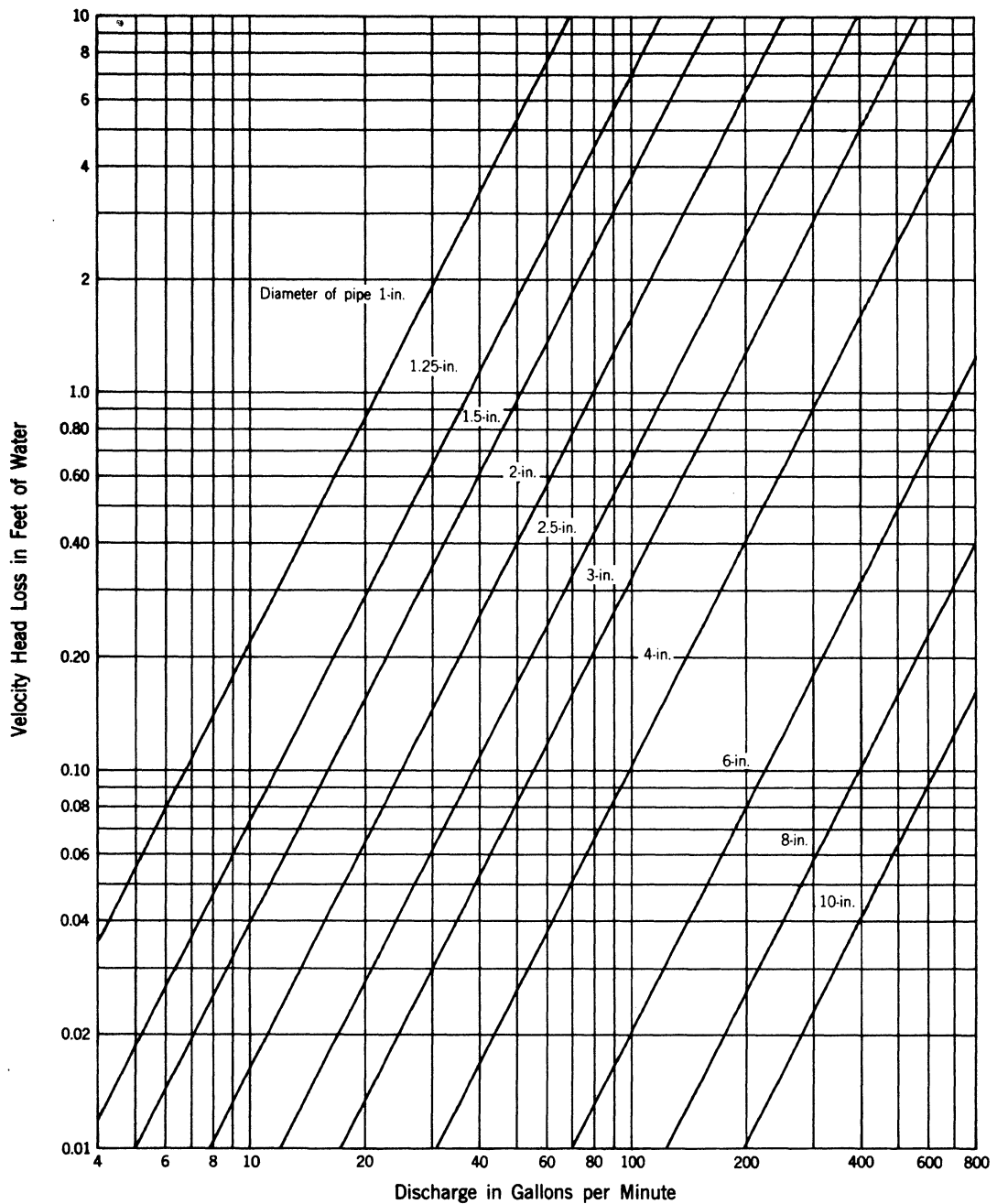


FRICTION LOSS IN WELL SCREENS AND RISER PIPES

HAZEN-WILLIAMS C=100

Note: Friction losses for value of C other than C = 100
multiply friction values by $(100/C)^{1.85}$

Figure A6



VELOCITY HEAD LOSS IN WELLS

Figure A7

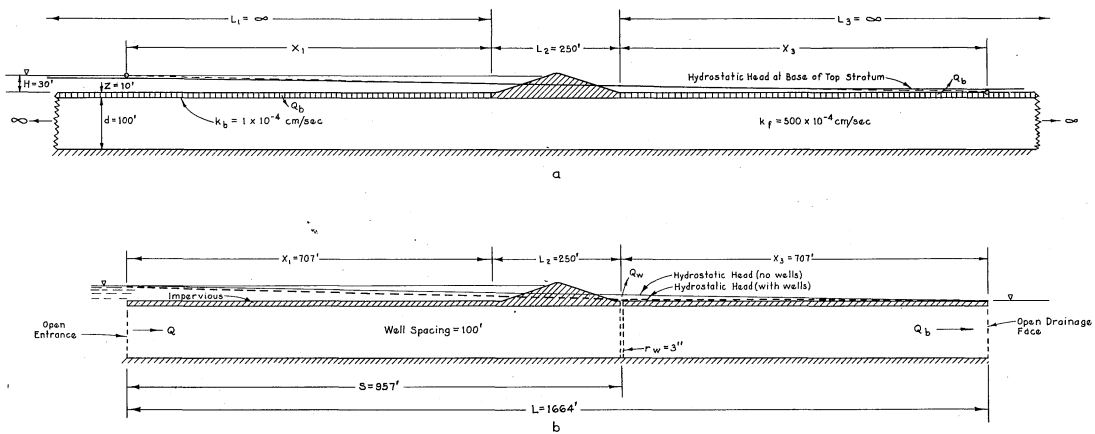
for the pervious stratum to obtain a third curve (dotted line) representing the head loss through both the pervious stratum and well, $h_1 + h_2$, for any given flow. When $h_1 = h_2$ equals the total head available ($H = 30$ ft in the example shown), the flow through the pervious stratum and well will be in equilibrium, and that flow represents the actual flow that would be obtained from the well. It should be emphasized that figure A5 represents only an example of how this problem in the design of a well system may be solved for a given well and a given pervious stratum. However, the principle illustrated may be used for any other well system or foundation.

18. In the example shown on figure A5, the well flow (Q_w) computed on the basis of no head losses within the well was 105 gpm. This well flow was reduced to 94 gpm as a result of the hydraulic head losses in the well. The head between the wells was increased by an amount equal to the well losses, or $P = 1.95 + h_2 = 1.95 + 2.50 = 4.45$ ft. This method of adjusting well flow for hydraulic head losses within the well is also approximately correct for well analyses where the landside top stratum is semi-pervious.

Case II -- Semi-pervious Top Stratum
Length of Riverside and Landside Top Stratum = ∞ ; Foundation
Infinite in Horizontal Extent; Full Penetration of
Pervious Stratum by Wells -- Bennett⁵ and Barron⁶

19. An approximate solution for well flow and residual pressure midway between wells for the condition shown in figure A8 may be obtained as follows. The case shown in figure A8-a may be transferred to that

5, 6 Op cit, page A2.



CASE II—SEMI-PERVIOUS TOP STRATUM INFINITE IN EXTENT

Figure A8

shown in figure A8-b (without wells) without changing the seepage volume or the hydrostatic head at the landside toe of the structure. This transformation is made by replacing the lengths of the leaking riverside and landside blankets by effective lengths of absolutely impervious blankets. The riverside blanket distance $L_1 = \infty$ may be reduced to an effective distance x_1 by:

$$x_1 = \frac{\tanh(L_1 c)}{c} \quad \dots \dots \dots (5)$$

where

$$c = \sqrt{\frac{k_b}{k_f z d}}$$

$$x_1 = \frac{\tanh \infty \sqrt{\frac{2 \times 10^{-4}}{1000 \times 10^{-4} \times 10 \times 100}}}{\sqrt{\frac{2 \times 10^{-4}}{1000 \times 10^{-4} \times 10 \times 100}}} = \frac{\tanh \infty}{\sqrt{\frac{2}{1,000,000}}} = \frac{1}{0.001414} = 707 \text{ ft.}$$

For this case $x_3 = x_1 = 707 \text{ ft}$

and $L = x_1 + L_2 + x_3 = 707 + 250 + 707 = 1664 \text{ ft.}$

It is pointed out that the riverside and landside top strata need not necessarily have the same characteristics in order to use the following formulas for well flow and head between wells. For this condition,

$$x_1 = \frac{\tanh (L_1 c_{\text{Riverside}})}{c_{\text{Riverside}}} = \frac{1}{c_{\text{Riverside}}} \quad (\text{Riverside}) \quad (5a)$$

$$x_3 = \frac{\tanh (L_3 c_{\text{Landside}})}{c_{\text{Landside}}} = \frac{1}{c_{\text{Landside}}} \quad (\text{Landside}) \quad (5b)$$

Where piezometer data are available at a given site in advance of design, the values of x_1 and x_3 may be obtained by projecting the straight-line portion of the hydraulic grade line beneath the levee or dam until it intersects the pool elevation on the riverside and the natural ground surface or tailwater on the landside. x_1 equals the distance from the riverside toe of the levee to the point of intersection with the pool level, and x_3 , the distance from the landside toe of the levee to the point of intersection with the tailwater or ground surface.

20. It is assumed that completely penetrating relief wells placed at the landside toe of the structure will have the same discharge rate and the same seepage pressure midway between wells beneath the blanket in figure A8-a as in figure A8-b. Thus, with wells installed, the total seepage Q_b through the upstream blanket equals Q in figure A8-b. Q_w is the well discharge in both cases, and Q_b (the seepage into the tailwater in fig. A8-b) is equal to the seepage through the landside blanket of figure A8-a. Thus in both figures $Q = Q_w + Q_b$.

21. Barron has obtained a solution for the conditions shown on figure A8-b with relief wells installed a distance "s" from the riverside entrance face of the pervious foundation and spaced at a distance of "a"

apart. The head between wells may be computed for this case from the following formula:

$$P = H \left[\frac{L - s}{L} \right] \left[1 - m^2 \right] \dots \dots \dots (6)$$

$$\text{where } m = \frac{1}{\log_e \frac{1}{2} \left(\frac{2L}{\pi r_w} \right)^2 + \log_e \left(1 - \cos \frac{2\pi s}{L} \right) + F} \dots \dots (7)$$

$$F = 2 \sum_{\substack{n = +\infty \\ n = 1, 2, 3 \\ \text{etc.}}} \log_e \left[\frac{\cosh \frac{\pi na}{L} - \cos \frac{2\pi s}{L}}{\cosh \frac{\pi na}{L} - 1} \right], \dots \dots (8)$$

$$f = \sum_{n = -\infty}^{n = +\infty} \log_e \left[\frac{\cosh \left(\frac{\pi a}{L} \right) (n - 1/2) - \cos \frac{2\pi s}{L}}{\cosh \frac{\pi a}{L} (n - 1/2) - 1} \right]. (9)$$

The values of F and f may be obtained from figures A9 and A10, respectively. The well discharge (Q_w) and seepage beyond the wells (Q_b) may be computed from the formulas:

$$Q_w = 4\pi H \left[\frac{L - s}{L} \right] m k_f d \dots \dots \dots (10)$$

$$Q_b = \frac{H a d k_f}{L} \left[1 - 4\pi \frac{s}{a} \left(\frac{L - s}{L} \right) m \right] \dots \dots \dots (11)$$

22. For the example shown in figure A8,

$$\frac{\pi a}{L} = \frac{100\pi}{1664} = 0.189; \quad \frac{s}{L} = \frac{957}{1664} = 0.575;$$

from figure A9, $F = 46$; from figure A10, $f = 52$; and

$$m = \frac{1}{\log_e \frac{1}{2} \left(\frac{2 \times 1664}{0.25 \pi} \right)^2 + \log_e \left(1 - \cos \frac{2\pi \times 957}{1664} \right) + 46} = 0.0159$$

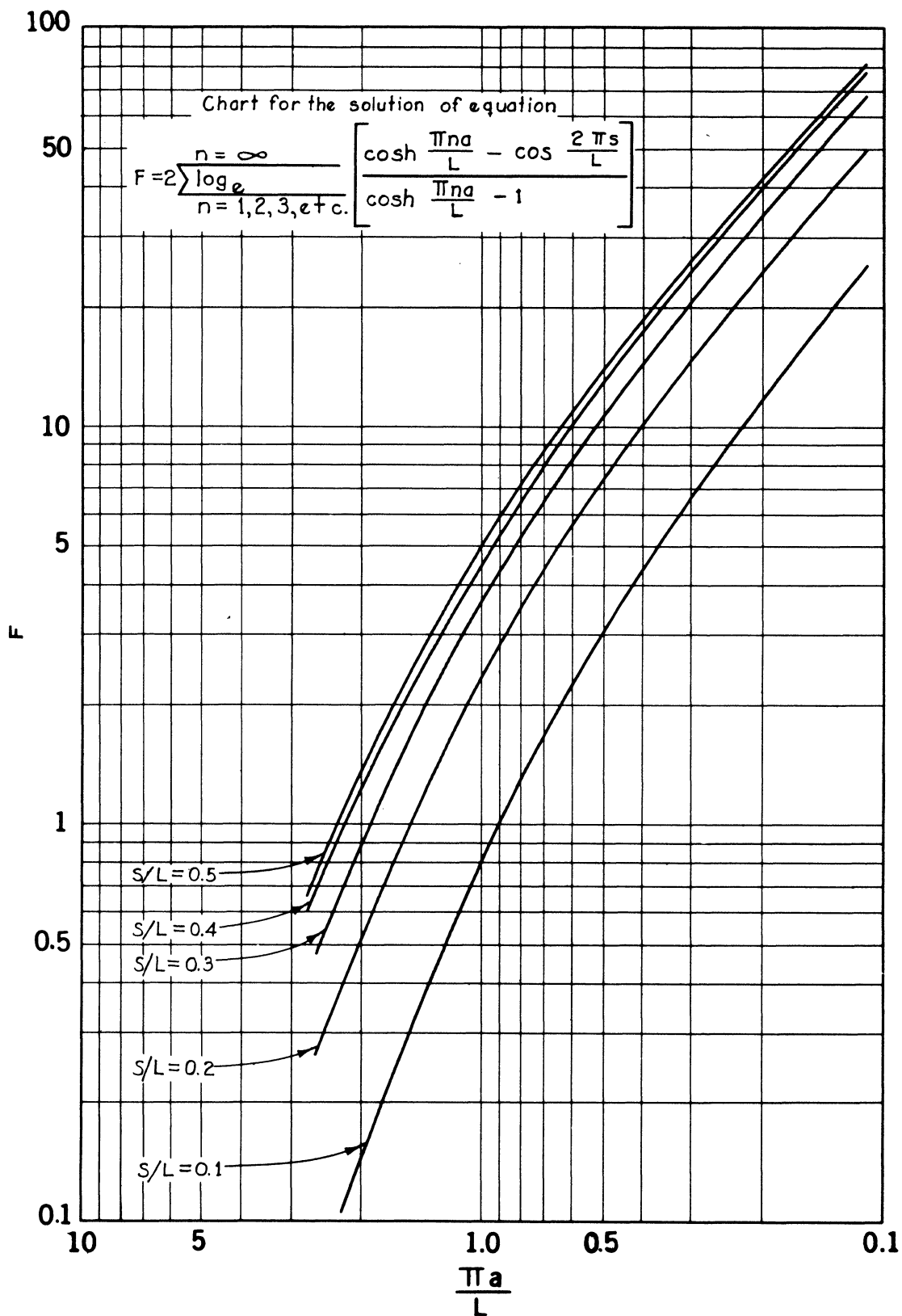


CHART FOR DETERMINATION OF F , EQUATION (8)

Figure A9

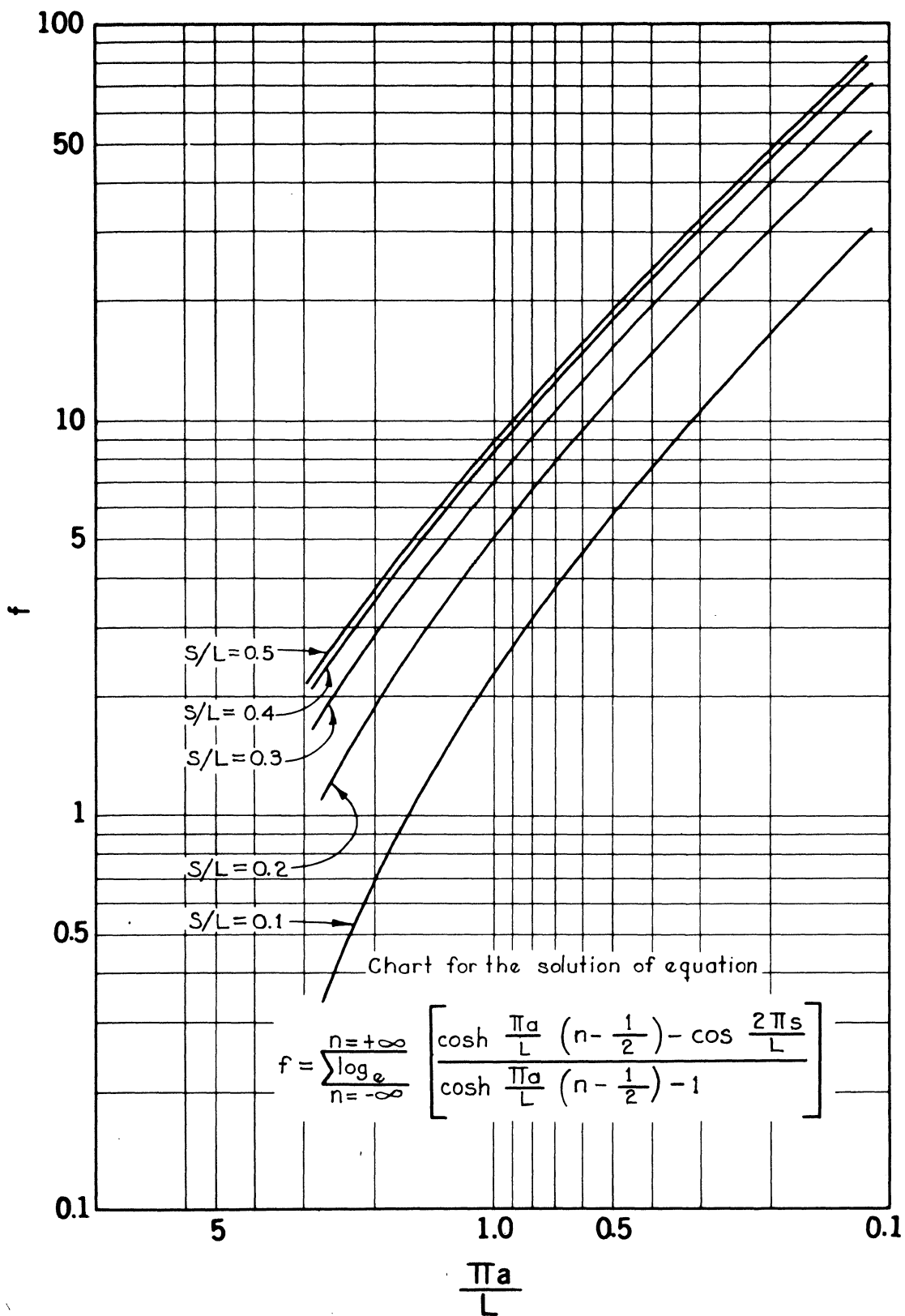


CHART FOR DETERMINATION OF f , EQUATION (9)

Figure A10

$$P = 30 \left[\frac{1664 - 957}{1664} \right] \left[1 - 0.0159 \times 52 \right] = 2.18 \text{ ft} = 7.26\% H$$

$$Q_w = 4\pi \cdot 30 \left[\frac{1664 - 957}{1664} \right] 0.0159 \times 0.10 \times 100 = 25.5 \text{ cfm} = 191 \text{ gpm} \\ = 6.36 H \text{ gpm.}$$

$$Q_b = \frac{30 \times 100 \times 100 \times 0.10}{1664} \left[1 - 4\pi \frac{957}{100} \left(\frac{1664 - 957}{1664} \right) 0.0159 \right] \\ = 3.38 \text{ cfm} = 25.3 \text{ gpm} = 0.88 H \text{ gpm,}$$

$$Q = Q_w + Q_b = 191 + 25 = 216 \text{ gpm or } 7.2 H \text{ gpm.}$$

It may be noted that the semi-pervious (sandy silt) riverside blanket in figure A8-a allowed as much water to enter the pervious foundation for the same well spacing and penetration as a vertical seepage entrance 1000 ft from the line of wells and an impervious riverside top stratum (fig. A3). (Compare flow values above with those given in paragraph 7 for Case I.)

23. The hydrostatic pressure at or landward of the landside levee toe without any relief wells may be computed for the case shown in figure A8-a from the following formula where x is the distance landward of the landside levee toe.

$$h_x = \frac{H e^{-cx}}{2 + c L_2} \quad \dots \dots \dots (12)$$

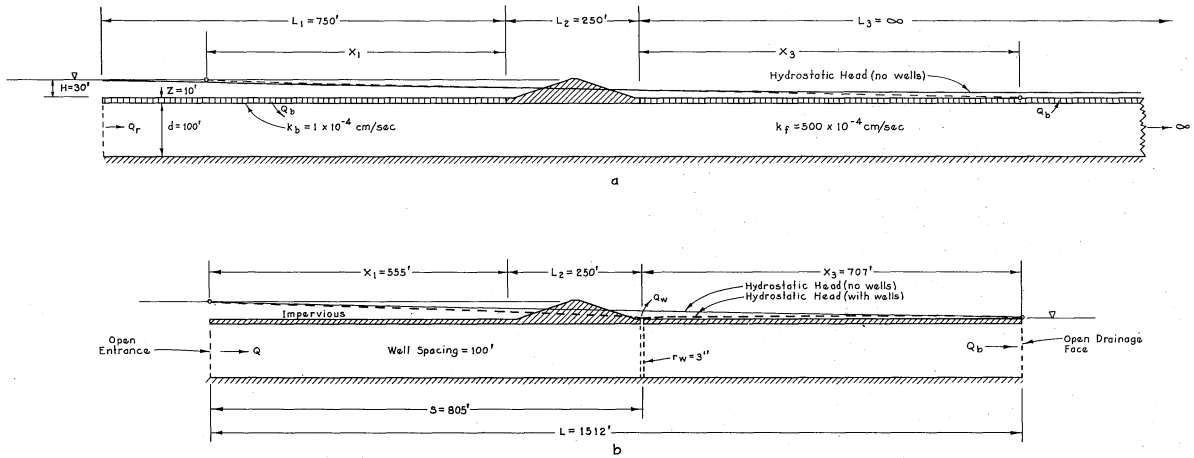
where $c = \sqrt{\frac{k_b}{k_f z d}}$

The natural seepage through the landside top stratum with no well system may be obtained from the following formula:

$$Q_b = \frac{H a d k_f}{L} \quad \dots \dots \dots (13)$$

Case III -- Semi-pervious Top Stratum
Distance from Wells to River = $(L_1 + L_2)$; Foundation Infinite in
Horizontal Extent Landward of Wells; Full Penetration of
Pervious Stratum of Wells -- Bennett and Barron

24. An approximate solution for well flow and residual pressure



CASE III-SEMI-PERVIOUS TOP STRATUM
WITH VERTICAL SEEPAGE ENTRANCE

Figure All

midway between wells for the condition shown in figure All may be obtained as follows. The case shown in figure All-a may be transferred to that shown in figure All-b (without wells) by replacing the lengths of the leaking riverside and landside blankets by effective lengths of impervious blankets. The riverside blanket distance $L_1 = 750$ ft may be reduced to an effective distance x_1 by:

$$x_1 = \frac{\tanh(c L_1)}{c} \quad \dots \dots \dots (14)$$

where $c = \sqrt{\frac{k_b}{k_f z d}}$

$$x_1 = \frac{\tanh 750 \sqrt{\frac{2 \times 10^{-4}}{1000 \times 10^{-4} \times 10 \times 100}}}{\sqrt{\frac{2 \times 10^{-4}}{1000 \times 10^{-4} \times 10 \times 100}}} = \frac{\tanh (750 \times 0.001414)}{0.001414} = 555 \text{ ft}$$

The landside blanket distance $L_3 = \infty$ may be reduced to an effective

distance x_3 by:

$$x_3 = \frac{1}{c} \dots \dots \dots (15)$$

$$x_3 = \frac{1}{\sqrt{\frac{2 \times 10^{-4}}{1000 \times 10^{-4} \times 10 \times 100}}} = \frac{1}{\sqrt{\frac{2}{1,000,000}}} = 707 \text{ ft.}$$

$$L = x_1 + L_2 + x_3 = 555 + 250 + 707 = 1512 \text{ ft.}$$

As pointed out in paragraph 19, where the riverside and landside top blanket are different, the thickness and permeability of the riverside blanket should be entered in the formula for x_1 , and the characteristics of the landside blanket in the formula for x_3 . This also applies for Case V which is discussed subsequently.

25. With the condition as shown in figure All-a converted to that in figure All-b, the head between the wells may be computed from equations 6-9, and the well flow and seepage beyond the wells from equations 10 and 11. For the example shown in figure All,

$$\frac{\pi a}{L} = \frac{100 \pi}{1512} = 0.208; \quad \frac{s}{L} = \frac{805}{1512} = 0.532;$$

from figure A9, $F = 41$; from figure A10, $f = 48$; and

$$m = \frac{1}{\log_e 1/2 \left(\frac{2 \times 1512}{\pi \times 0.25} \right)^2 + \log_e \left(1 - \cos \frac{2 \pi 805}{1512} \right) + 41} = 0.0174$$

$$P = 30 \left[\frac{1512 - 805}{1512} \right] \left[1 - 0.0174 \times 48 \right] = 2.30 \text{ ft} = 7.66\% H$$

$$Q_w = 4 \pi 30 \left[\frac{1512 - 805}{1512} \right] 0.0174 \times 0.10 \times 100 = 30.6 \text{ cfm} = 229 \text{ gpm} \\ = 7.61 \text{ H gpm}$$

$$Q_b = \frac{30 \times 100 \times 100 \times 0.10}{1512} \left[1 - 4\pi \frac{805}{100} \left(\frac{1512 - 805}{1512} \right) 0.0174 \right]$$

$$= 3.49 \text{ cfm} = 26.1 \text{ gpm} = 0.87 \text{ H gpm.}$$

$$Q = Q_w + Q_b = 229 + 26 = 255 \text{ gpm.}$$

26. The hydrostatic pressure at or landward of the landside levee toe without any relief wells may be computed for the case shown in figure All-a from the following formula.

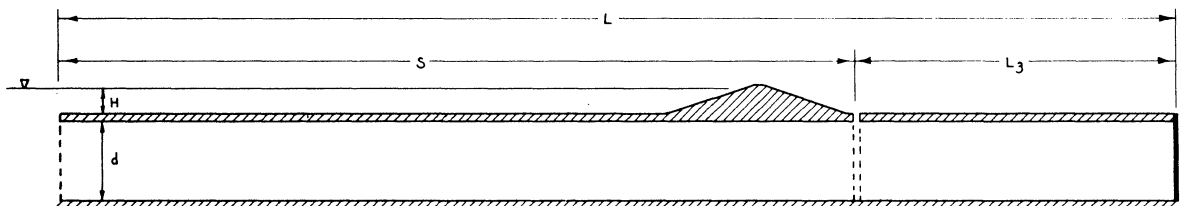
$$h_x = \frac{H e^{-cx}}{1 + \tanh(c L_1) + c L_2} \quad \dots \dots \dots (16)$$

The natural seepage landward of the levee without any wells would be:

$$Q_b = \frac{H a d k_f}{L} \quad \dots \dots \dots (17)$$

Case IV -- Impervious Top Stratum
Distance from Wells to River = s ; Pervious Foundation
Blocked Landward at a Distance L_3 from Wells;
Full Penetration of Pervious Stratum by Wells -- Barron

27. A solution for the residual head between the wells for this case (see fig. A12) may be obtained from the following formula:



CASE IV-IMPERVIOUS TOP STRATUM
PERVIOUS FOUNDATION BLOCKED ON LANDSIDE

Figure A12

$$P = H + V \sum_{n=-\infty}^{n=+\infty} \log_e \left[\left(\frac{\cosh \frac{\pi a}{2L} (n - \frac{1}{2}) - \cos \frac{\pi s}{L}}{\cosh \frac{\pi a}{2L} (n - \frac{1}{2}) - 1} \right) \left(\frac{\cosh \frac{\pi a}{2L} (n - \frac{1}{2}) - \cos \pi}{\cosh \frac{\pi a}{2L} (n - \frac{1}{2}) - \cos \frac{\pi}{L} (s - L)} \right) \right] \dots \dots \dots (18)$$

where

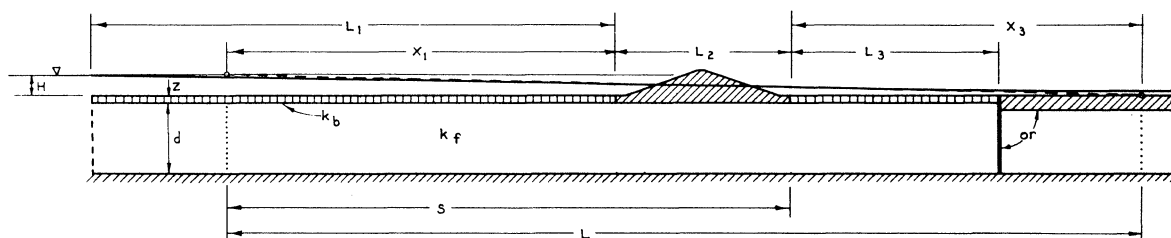
$$V = \frac{-H}{\sum_{n=-\infty}^{n=+\infty} \log_e \left[\left(\frac{\cosh \frac{\pi na}{2L} - \cos \frac{\pi s}{L}}{\cosh \frac{\pi na}{2L} - \cos \frac{\pi r_w}{2L}} \right) \left(\frac{\cosh \frac{\pi na}{2L} + 1}{\cosh \frac{\pi na}{2L} + \cos \frac{\pi s}{L}} \right) \right]}$$

No formulas are available for the well flow.

Case V -- Semi-pervious Top Stratum
Distance from Wells to River = $L_1 + L_2$; Pervious Foundation
Blocked at a Distance L_3 from Wells; Full Penetration
of Pervious Stratum by Wells -- Barron and Bennett

28. Where the top stratum is semi-pervious, with the other conditions being the same as in Case IV, a solution for the head between the wells may be obtained by transferring the lengths of riverside and land-side top strata to equivalent lengths of impervious blankets and then using the formula given for Case IV. For this case,

$$s = x_1 + L_2, \quad \text{and} \quad L = x_1 + L_2 + x_3$$



CASE V- SEMI-PERVIOUS TOP STRATUM
PERVIOUS FOUNDATION BLOCKED ON LANDSIDE

Figure A13

$$x_1 = \frac{\tanh (c L_1)}{c} \quad \dots \dots \dots (19)$$

$$x_3 = \frac{1}{\tanh (c L_3)} \quad \dots \dots \dots (20)$$

29. The hydrostatic pressure at the landside levee toe, and at the distance L_3 from the levee toe, with no well system may be obtained from the following formulas:

$$h_{x=0} = \frac{H x_3}{L} \quad \dots \dots \dots (21)$$

and

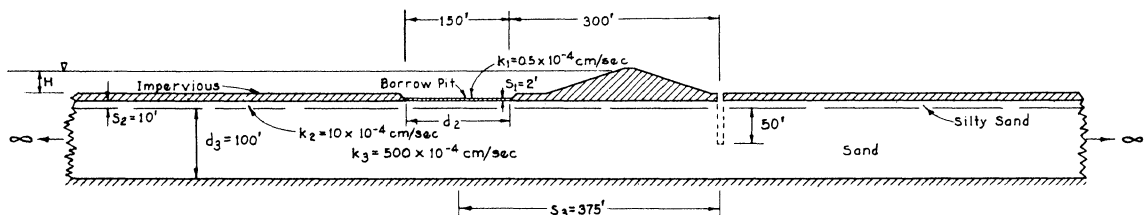
$$h_{x=L_3} = \frac{H x_3}{L \cosh (c L_3)} \quad \dots \dots \dots (22)$$

The natural seepage landward of the levee without any wells would be:

$$Q_b = \frac{H a d k_f}{L} \quad \dots \dots \dots (23)$$

Case VI -- Open Riverside Borrow Pits (An Approximate Solution)

30. An approximate method for determining well flow and pressure midway between wells for the case of an impervious top stratum and open riverside borrow pits is illustrated by the example in figure A14 below.



CASE VI- OPEN RIVERSIDE BORROW PITS

Figure A14

If the flow downward through the bottom of the borrow pit and the upper stratum of silty sand is assumed to be vertical, the distances s_1 and s_2 may be converted into equivalent lengths of foundation (with $d_3 = 100$ and $k_3 = 500 \times 10^{-4}$ cm/sec). In the above example, $Q_1 = Q_2 = Q_3 = Q_w$. Replacing (1) and (2) with an equivalent length of (3) (see fig. A14),

$$s_{3-(1)} = \frac{k_3}{k_1} \times \frac{d_3}{d_1} \times s_1 = \frac{500 \times 10^{-4}}{0.5 \times 10^{-4}} \times \frac{100}{150} \times 2 = 1333 \text{ ft}$$

$$s_{3-(2)} = \frac{k_3}{k_2} \times \frac{d_3}{d_2} \times s_2 = \frac{500 \times 10^{-4}}{10 \times 10^{-4}} \times \frac{100}{150} \times 10 = 333 \text{ ft}$$

The effective distance from the line of wells to the seepage entrance, on the basis of an impervious top stratum and assuming all horizontal flow to be carried by the more pervious foundation stratum, would be

$$s = s_{3-(1)} + s_{3-(2)} + s_3 = 1333 + 333 + 375 = 2041 \text{ ft}$$

The well flow and head between wells can then be computed from figure A4. For a well spacing of $a = 100$ ft and $H = 30$ ft, in the above example,

$$Q_w = 13.0 \text{ cfm} = 97 \text{ gpm} = 3.24 H \text{ gpm}$$

$$\text{and } P = 3.81 \text{ ft} = 12.7\% H.$$